

STRUCTURAL CALCULATIONS FOR :

Paragon Star Restroom Concession Lee's Summit, Missouri

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Summary

Loads for the project referenced above were determined based on the governing International Building Code (IBC) and the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7).

All vertical/gravity loads were determined as follow: All dead loads were determined based on the building composition and all live loads were determined based on the expected occupancy for each of the spaces within the building. Snow loads were determined based on the building dimensions, the roof profile and the project location.

All lateral loads were determined as follows: All wind loading was based on the building dimensions and project location. All seismic loads were determined based on the building composition, the type of lateral stability system and the project location.

The following section of calculations covers the process used to determine the gravity and lateral loads for the project referenced above. Refer to all other sections for the application of these loads.

DEAD LOADING

Roof Dead Load

Material	Thickness (in)	γ (lb/ft³)	Weight (lb/ft²)
Roofing Material (60 MIL TPO)	0		0.5
Poly-ISO Insulation (R-25)	Varies		2.0
3/4" Plywood Decking	0.75	38.4	2.5
Wood Trusses			5.0
MEP			2.5
Collateral			2.5
Totals			15.0

LIVE LOADING

LIVE LOAD CONSTRUCTION

Roof Live Load

20.0 psf

Wall Loads

2x6 Wood Studs w/ Metal Panels

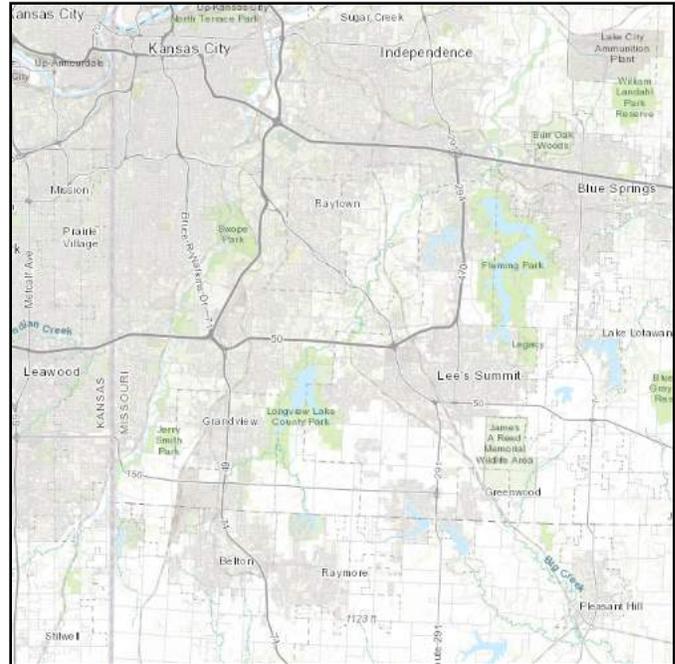
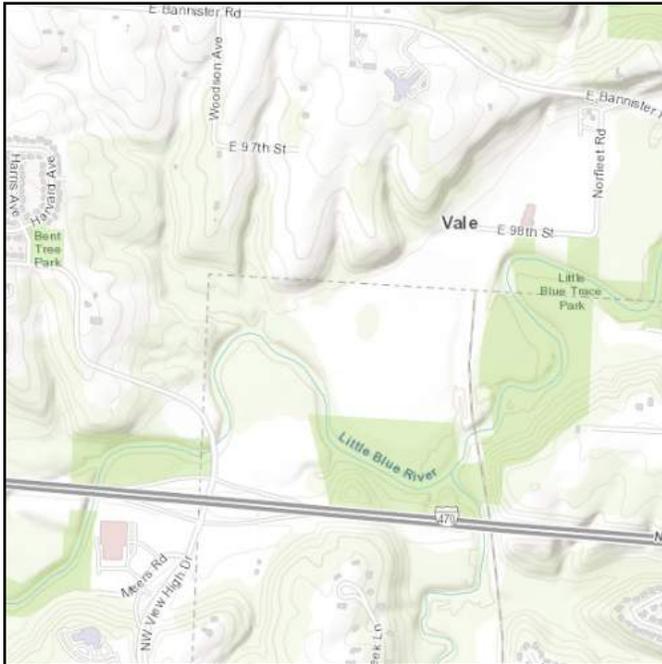
Material	Thickness (in)	γ (lb/ft³)	Weight (lb/ft²)
2x6 Studs with 5/8" gyp.	5.5		8.5
Metal Wall Panel	2.0		2.0
7/16" Plywood Sheathing	0.4375		1.5
Totals			12psf

ASCE Hazards Report

Address:
Paragon Star Sports Complex
- 1401 NW River Rd
Lees Summit,

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 38.9409
Longitude: -94.4433
Elevation: 810.1759441441334 ft
(NAVD 88)



Wind

Results:

Wind Speed	109 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Fri Jul 25 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.



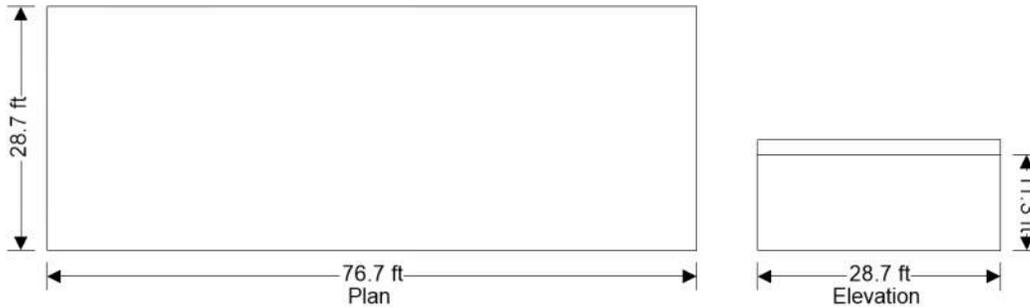
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WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.19



Building data

Type of roof	Flat
Length of building	b = 76.67 ft
Width of building	d = 28.67 ft
Height to eaves	H = 11.25 ft
Height of parapet	h _p = 1.75 ft
Mean height	h = 11.25 ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = 3.00 ft

General wind load requirements

Basic wind speed	V = 109.0 mph
Risk category	II
Wind directionality factor (Table 26.6-1)	K _d = 0.85
Ground elevation above sea level	Z _{gl} = 810 ft
Ground elevation factor	K _e = exp(-0.0000362 × Z _{gl} /1ft) = 0.97
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC _{pi_p} = 0.18
Internal pressure coef -ve (Table 26.13-1)	GC _{pi_n} = -0.18
Parapet enclosure classification	Enclosed
Parapet internal pressure coef +ve (Table 26.13-1)	GC _{pi_pp} = 0.18
Parapet internal pressure coef -ve (Table 26.13-1)	GC _{pi_np} = -0.18
Gust effect factor	G _f = 1.01

Topography

Topography factor not significant	K _{zt} = 1.0
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Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K _z = 0.85
Velocity pressure	q _h = 0.00256 × K _z × K _{zt} × K _d × K _e × V ² × 1psf/mph ² = 21.3 psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K _z = 0.85
Velocity pressure	q _p = 0.00256 × K _z × K _{zt} × K _d × K _e × V ² × 1psf/mph ² = 21.3 psf



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Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_i = 21.34$ psf

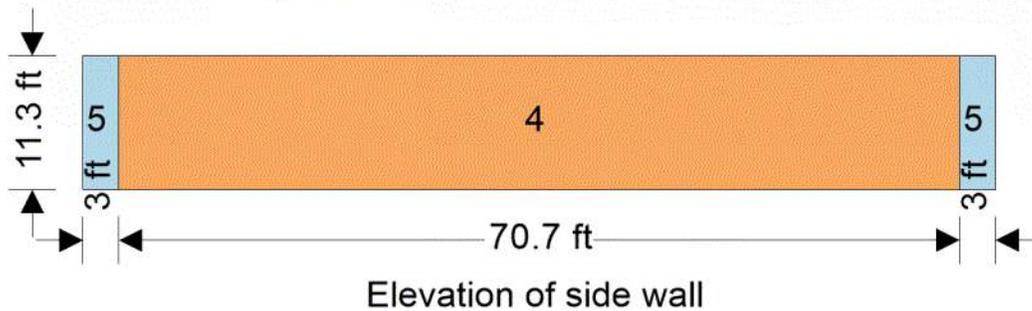
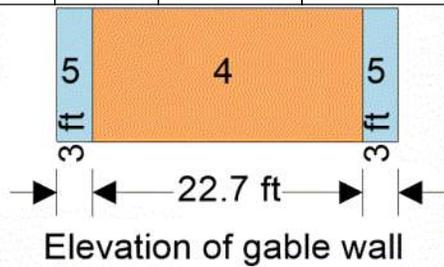
Equations used in tables

Net pressure $p = q_h \times (GC_p - GC_{pi})$

Parapet net pressure $p = q_p \times (GC_p - GC_{pi_p})$

Components and cladding pressures - Wall (Table 30.3-1 and (Figure 30.3-2A))

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft ²)	+GC _p	-GC _p	Pres (+ve) (psf)	Pres (-ve) (psf)
<10 sf	4	-	-	10.0	0.90	-0.99	23.0	-25.0
20 sf	4	-	-	20.0	0.85	-0.94	22.0	-23.9
50 sf	4	-	-	50.0	0.79	-0.88	20.7	-22.6
>100 sf	4	-	-	100.0	0.74	-0.83	19.7	-21.6
>500 sf	4	-	-	500.0	0.63	-0.72	17.3	-19.2
<10 sf	5	-	-	10.0	0.90	-1.26	23.0	-30.7
20 sf	5	-	-	20.0	0.85	-1.16	22.0	-28.7
50 sf	5	-	-	50.0	0.79	-1.04	20.7	-26.0
>100 sf	5	-	-	100.0	0.74	-0.94	19.7	-23.9
>500 sf	5	-	-	500.0	0.63	-0.72	17.3	-19.2
10sf (W)	5p	-	-	10.0	0.90	-2.30	23.0	-52.9



Components and cladding pressures - Roof (Figure 30.3-2A)

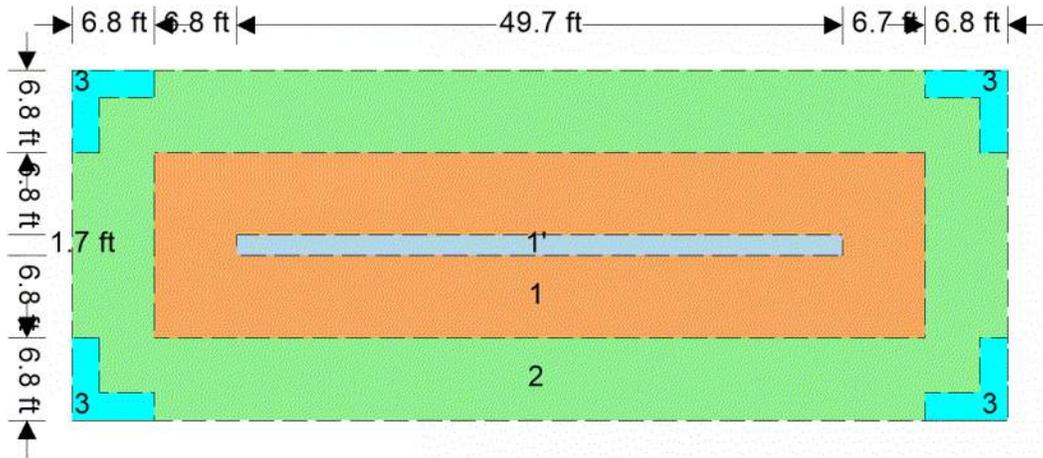
Component	Zone	Length (ft)	Width (ft)	Eff. area (ft ²)	+GC _p	-GC _p	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	10.2 #	-40.1
20 sf	1	-	-	20.0	0.27	-1.58	9.6 #	-37.5



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Component	Zone	Length (ft)	Width (ft)	Eff. area (ft ²)	+GC _p	-GC _p	Pres (+ve) (psf)	Pres (-ve) (psf)
50 sf	1	-	-	50.0	0.23	-1.41	8.8 #	-34.0
>100 sf	1	-	-	100.0	0.20	-1.29	8.1 #	-31.3
<=10 sf	1'	-	-	10.0	0.30	-0.90	10.2 #	-23.0
20 sf	1'	-	-	20.0	0.27	-0.90	9.6 #	-23.0
50 sf	1'	-	-	50.0	0.23	-0.90	8.8 #	-23.0
>100 sf	1'	-	-	100.0	0.20	-0.90	8.1 #	-23.0
<=10 sf	2	-	-	10.0	0.30	-2.30	10.2 #	-52.9
20 sf	2	-	-	20.0	0.27	-2.14	9.6 #	-49.5
50 sf	2	-	-	50.0	0.23	-1.93	8.8 #	-45.0
>100 sf	2	-	-	100.0	0.20	-1.77	8.1 #	-41.6
<=10 sf	3	-	-	10.0	0.30	-3.20	10.2 #	-72.1
20 sf	3	-	-	20.0	0.27	-2.88	9.6 #	-65.3
50 sf	3	-	-	50.0	0.23	-2.46	8.8 #	-56.3
>100 sf	3	-	-	100.0	0.20	-2.14	8.1 #	-49.5

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof



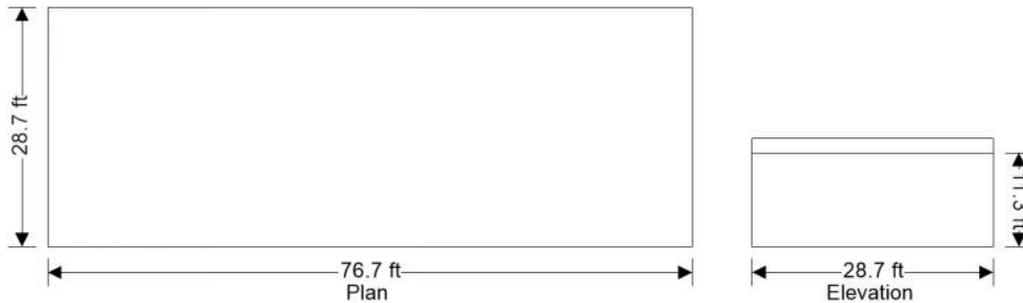
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WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.19



Building data

Type of roof	Flat
Length of building	b = 76.67 ft
Width of building	d = 28.67 ft
Height to eaves	H = 11.25 ft
Height of parapet	h _p = 1.75 ft
Mean height	h = 11.25 ft
End zone width	a = max(min(0.1 × min(b, d), 0.4 × h), 0.04 × min(b, d), 3ft) = 3.00 ft
Plan length of Zone 2/2E when GC _{pf} negative	L _{z2} = min(0.5 × d, 2.5 × H) = 14.34 ft
Plan length of Zone 3/3E encroachment on zone 2	L _{z3} = max(0 ft, 0.5 × d - L _{z2}) = 0.00 ft

General wind load requirements

Basic wind speed	V = 109.0 mph
Risk category	II
Wind directionality factor (Table 26.6-1)	K _d = 0.85
Ground elevation above sea level	z _{gl} = 810 ft
Ground elevation factor	K _e = exp(-0.0000362 × z _{gl} /1ft) = 0.97
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC _{pi,p} = 0.18
Internal pressure coef -ve (Table 26.13-1)	GC _{pi,n} = -0.18

Topography

Topography factor not significant	K _{zt} = 1.0
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Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K _z = 0.85
Velocity pressure	q _h = 0.00256 × K _z × K _{zt} × K _d × K _e × V ² × 1psf/mph ² = 21.3 psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K _z = 0.85
Velocity pressure	q _p = 0.00256 × K _z × K _{zt} × K _d × K _e × V ² × 1psf/mph ² = 21.3 psf

Parapet pressures and forces

Velocity pressure at top of parapet	q _p = 21.34 psf
Combined net pressure coefficient, leeward	GC _{pnl} = -1.0
Combined net parapet pressure, leeward	p _{pl} = q _p × GC _{pnl} = -21.34 psf



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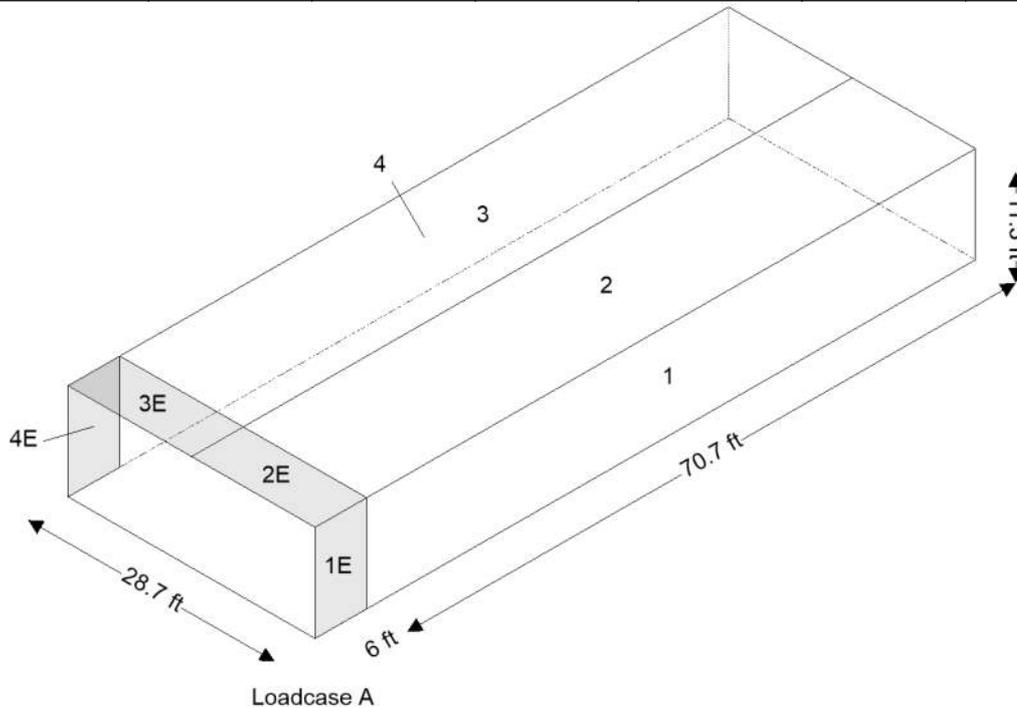
Combined net pressure coefficient, windward $GC_{pnw} = 1.5$
 Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw} = 32.01$ psf
 Wind direction 0 deg (|| to width):
 Leeward parapet force $F_{w,wpl_0} = p_{pl} \times h_p \times b = -2.9$ kips
 Windward parapet force $F_{w,wpw_0} = p_{pw} \times h_p \times b = 4.3$ kips
 Wind direction 90 deg (|| to length):
 Leeward parapet force $F_{w,wpl_90} = p_{pl} \times h_p \times d = -1.1$ kips
 Windward parapet force $F_{w,wpw_90} = p_{pw} \times h_p \times d = 1.6$ kips

Design wind pressures

Design wind pressure equation $p = q_h \times ((GC_{pf}) - (GC_{pi}))$

Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GC_{pi})}$ (psf)	$p_{(-GC_{pi})}$ (psf)	Area (ft ²)	+F _{wi} (kips)	-F _{wi} (kips)
1	0.40	4.7	12.4	795	3.7	9.8
2	-0.69	-18.6	-10.9	1013	-18.8	-11.0
3	-0.37	-11.7	-4.1	1013	-11.9	-4.1
4	-0.29	-10.0	-2.3	795	-8.0	-1.9
1E	0.61	9.2	16.9	68	0.6	1.1
2E	-1.07	-26.7	-19.0	86	-2.3	-1.6
3E	-0.53	-15.2	-7.5	86	-1.3	-0.6
4E	-0.43	-13.0	-5.3	68	-0.9	-0.4



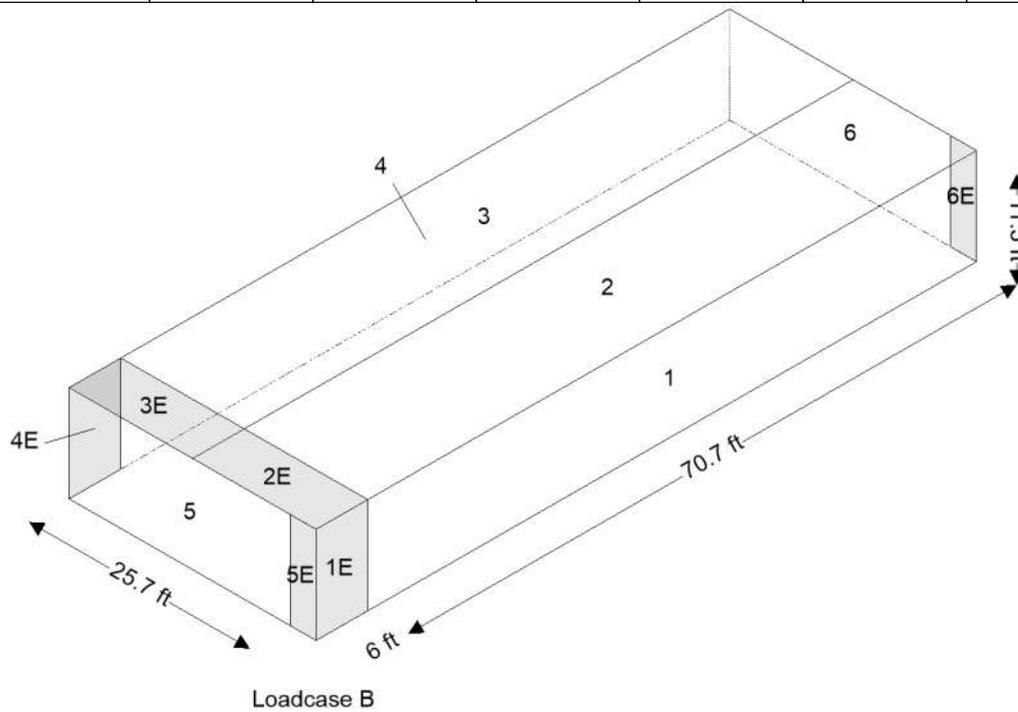
Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GC_{pi})}$ (psf)	$p_{(-GC_{pi})}$ (psf)	Area (ft ²)	+F _{wi} (kips)	-F _{wi} (kips)
1	-0.45	-13.4	-5.8	795	-10.7	-4.6



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2	-0.69	-18.6	-10.9	1013	-18.8	-11.0
3	-0.37	-11.7	-4.1	1013	-11.9	-4.1
4	-0.45	-13.4	-5.8	795	-10.7	-4.6
5	0.40	4.7	12.4	289	1.4	3.6
6	-0.29	-10.0	-2.3	289	-2.9	-0.7
1E	-0.48	-14.1	-6.4	68	-1.0	-0.4
2E	-1.07	-26.7	-19.0	86	-2.3	-1.6
3E	-0.53	-15.2	-7.5	86	-1.3	-0.6
4E	-0.48	-14.1	-6.4	68	-1.0	-0.4
5E	0.61	9.2	16.9	34	0.3	0.6
6E	-0.43	-13.0	-5.3	34	-0.4	-0.2



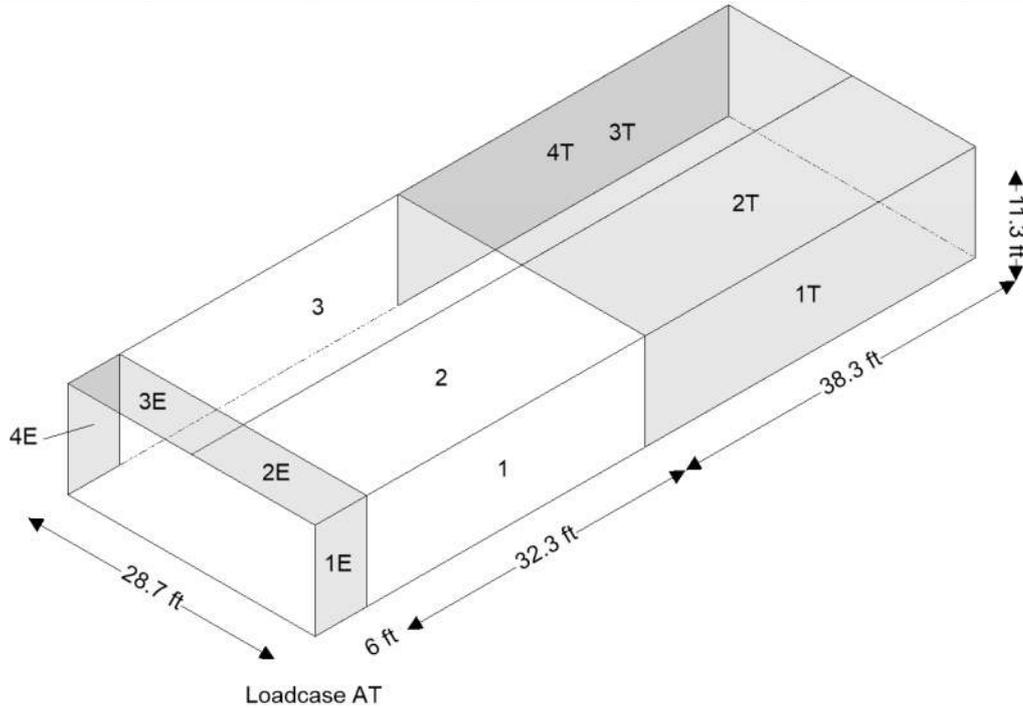
Design wind pressures – Loadcase AT

Zone	GC _{pf}	p(+GC _{pi}) (psf)	p(-GC _{pi}) (psf)	Area (ft ²)	+F _{wi} (kips)	-F _{wi} (kips)
1	0.40	4.7	12.4	364	1.7	4.5
2	-0.69	-18.6	-10.9	464	-8.6	-5.0
3	-0.37	-11.7	-4.1	464	-5.4	-1.9
4	-0.29	-10.0	-2.3	364	-3.6	-0.9
1E	0.61	9.2	16.9	68	0.6	1.1
2E	-1.07	-26.7	-19.0	86	-2.3	-1.6
3E	-0.53	-15.2	-7.5	86	-1.3	-0.6
4E	-0.43	-13.0	-5.3	68	-0.9	-0.4
1T	-	1.2	3.1	431	0.5	1.3



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2T	-	-4.6	-2.7	550	-2.6	-1.5
3T	-	-2.9	-1.0	550	-1.6	-0.6
4T	-	-2.5	-0.6	431	-1.1	-0.3

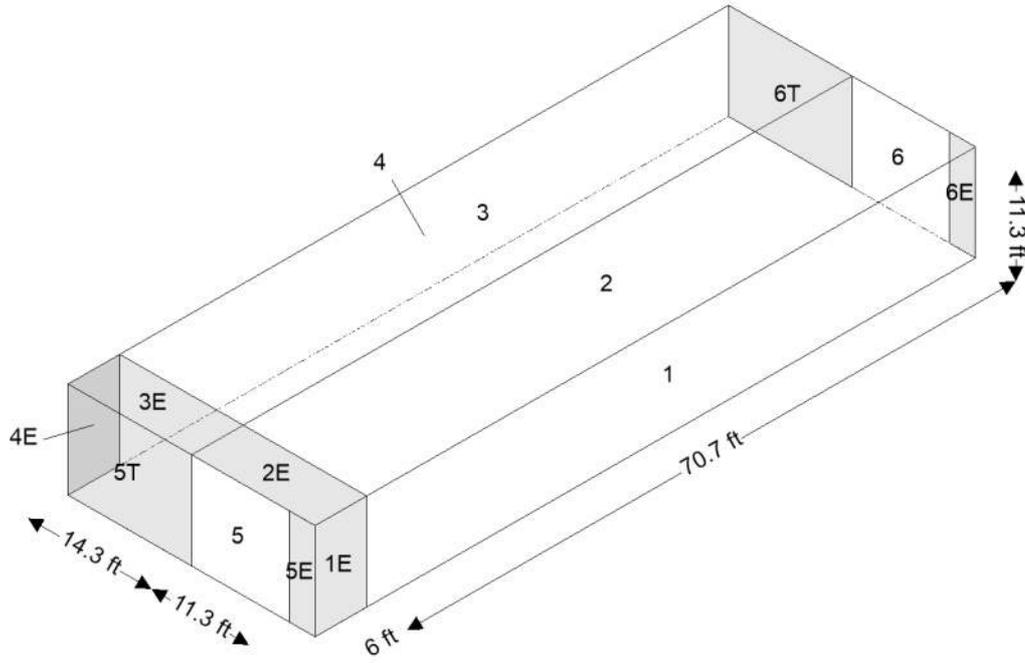


Design wind pressures – Loadcase BT

Zone	GC _{pf}	p(+GC _{pi}) (psf)	p(-GC _{pi}) (psf)	Area (ft ²)	+F _{wi} (kips)	-F _{wi} (kips)
1	-0.45	-13.4	-5.8	795	-10.7	-4.6
2	-0.69	-18.6	-10.9	1013	-18.8	-11.0
3	-0.37	-11.7	-4.1	1013	-11.9	-4.1
4	-0.45	-13.4	-5.8	795	-10.7	-4.6
5	0.40	4.7	12.4	144	0.7	1.8
6	-0.29	-10.0	-2.3	144	-1.4	-0.3
1E	-0.48	-14.1	-6.4	68	-1.0	-0.4
2E	-1.07	-26.7	-19.0	86	-2.3	-1.6
3E	-0.53	-15.2	-7.5	86	-1.3	-0.6
4E	-0.48	-14.1	-6.4	68	-1.0	-0.4
5E	0.61	9.2	16.9	17	0.2	0.3
6E	-0.43	-13.0	-5.3	34	-0.4	-0.2
5T	-	1.2	3.1	161	0.2	0.5
6T	-	-2.5	-0.6	161	-0.4	-0.1



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Loadcase BT

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft ²
Mapped Elevation:	810.2 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Fri Jul 25 2025

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

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SNOW LOAD - LONG DIRECTION
SNOW LOADING
In accordance with ASCE7-16

Tedds calculation version 1.0.12

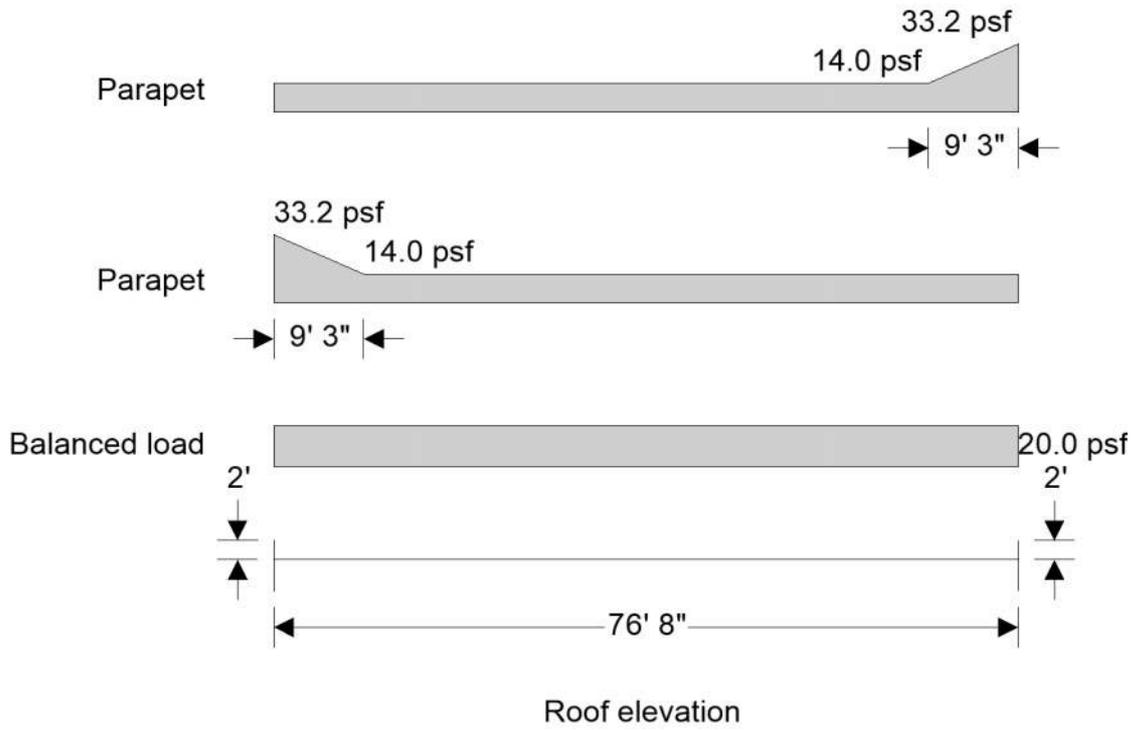
Building details

 Roof type Flat
 Width of roof $b = 76.67$ ft
Ground snow load

 Ground snow load (Figure 7.2-1) $p_g = 20.00$ lb/ft²
 Density of snow $\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = 16.60$ lb/ft³
 Surface roughness category (Sect. 26.7) C
 Exposure condition (Table 7.3-1) Partially exposed
 Exposure factor (Table 7.3-1) $C_e = 1.00$
 Thermal condition (Table 7.3-2) All
 Thermal factor (Table 7.3-2) $C_t = 1.00$
 Importance category (Table 1.5-1) II
 Importance factor (Table 1.5-2) $I_s = 1.00$
 Min snow load for low slope roofs (Sect 7.3.4) $p_{f_min} = I_s \times p_g = 20.00$ lb/ft²
 Flat roof snow load (Sect 7.3) $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 14.00$ lb/ft²
Left parapet

 Balanced snow load height $h_b = p_f / \gamma = 0.84$ ft
 Height of left parapet $h_{pptL} = 2.00$ ft
 Height from balance load to top of left parapet $h_{c_pptL} = h_{pptL} - h_b = 1.16$ ft
 Length of roof - left parapet $l_{u_pptL} = b = 76.67$ ft
 Drift height windward drift - left parapet $h_{d_l_pptL} = \sqrt{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})} = 2.08$ ft
 Drift height - left parapet $h_{d_pptL} = \min(h_{d_l_pptL}, h_{pptL} - h_b) = 1.16$ ft
 Drift width $W_{d_pptL} = \min(4 \times h_{d_l_pptL}^2 / h_{c_pptL}, 8 \times (h_{pptL} - h_b), b) = 9.25$ ft
 Drift surcharge load - left parapet $p_{d_pptL} = h_{d_pptL} \times \gamma = 19.20$ lb/ft²
Right parapet

 Height of right parapet $h_{pptR} = 2.00$ ft
 Height from balance load to top of right parapet $h_{c_pptR} = h_{pptR} - h_b = 1.16$ ft
 Length of roof - right parapet $l_{u_pptR} = b = 76.67$ ft
 Drift height windward drift - right parapet $h_{d_l_pptR} = \sqrt{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})} = 2.08$ ft
 Drift height - right parapet $h_{d_pptR} = \min(h_{d_l_pptR}, h_{pptR} - h_b) = 1.16$ ft
 Drift width $W_{d_pptR} = \min(4 \times h_{d_l_pptR}^2 / h_{c_pptR}, 8 \times (h_{pptR} - h_b), b) = 9.25$ ft
 Drift surcharge load - right parapet $p_{d_pptR} = h_{d_pptR} \times \gamma = 19.20$ lb/ft²



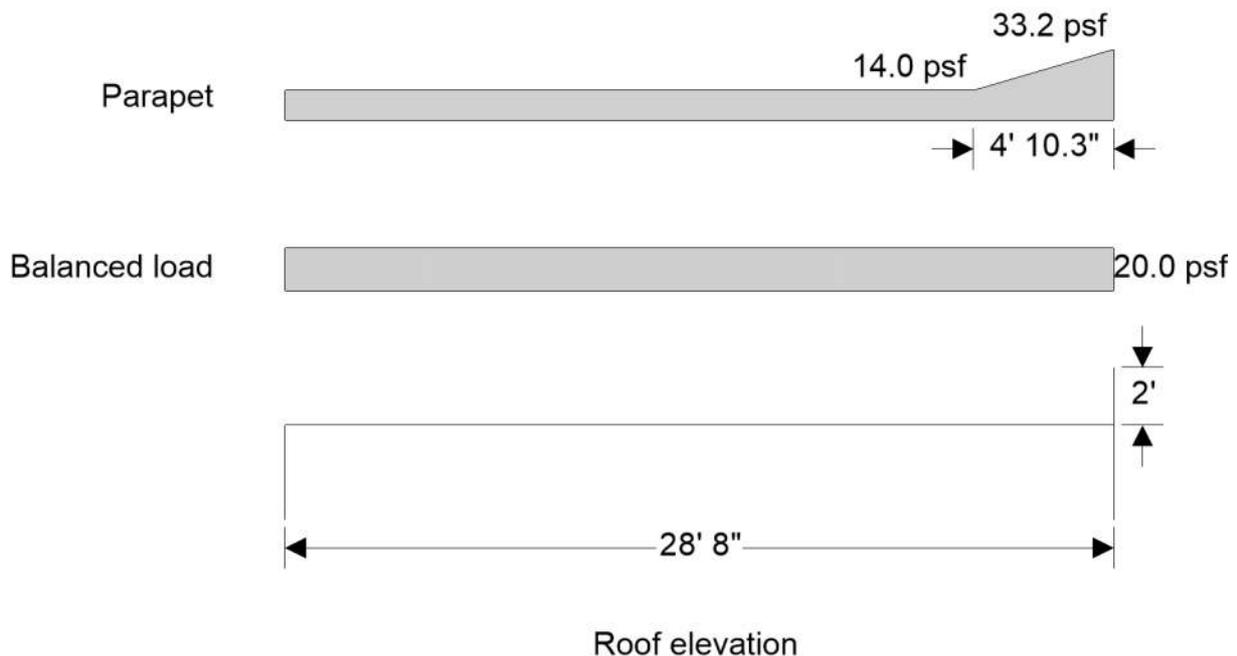
SNOW LOAD - SHORT DIRECTION
SNOW LOADING
In accordance with ASCE7-16

Tedds calculation version 1.0.12

Building details

 Roof type **Flat**
 Width of roof **b = 28.67 ft**
Ground snow load

 Ground snow load (Figure 7.2-1) **$p_g = 20.00 \text{ lb/ft}^2$**
 Density of snow **$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = 16.60 \text{ lb/ft}^3$**
 Surface roughness category (Sect. 26.7) **C**
 Exposure condition (Table 7.3-1) **Partially exposed**
 Exposure factor (Table 7.3-1) **$C_e = 1.00$**
 Thermal condition (Table 7.3-2) **All**
 Thermal factor (Table 7.3-2) **$C_t = 1.00$**
 Importance category (Table 1.5-1) **II**
 Importance factor (Table 1.5-2) **$I_s = 1.00$**
 Min snow load for low slope roofs (Sect 7.3.4) **$p_{f_min} = I_s \times p_g = 20.00 \text{ lb/ft}^2$**
 Flat roof snow load (Sect 7.3) **$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 14.00 \text{ lb/ft}^2$**
Right parapet

 Balanced snow load height **$h_b = p_f / \gamma = 0.84 \text{ ft}$**
 Height of right parapet **$h_{pptR} = 2.00 \text{ ft}$**
 Height from balance load to top of right parapet **$h_{c_pptR} = h_{pptR} - h_b = 1.16 \text{ ft}$**
 Length of roof - right parapet **$l_{u_pptR} = b = 28.67 \text{ ft}$**
 Drift height windward drift - right parapet **$h_{d_l_pptR} = \sqrt{(I_s) \times 0.75 \times (0.43 \times (\max(20 \text{ ft}, l_{u_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})} = 1.19 \text{ ft}$**
 Drift height - right parapet **$h_{d_pptR} = \min(h_{d_l_pptR}, h_{pptR} - h_b) = 1.16 \text{ ft}$**
 Drift width **$W_{d_pptR} = \min(4 \times h_{d_l_pptR}^2 / h_{c_pptR}, 8 \times (h_{pptR} - h_b), b) = 4.86 \text{ ft}$**
 Drift surcharge load - right parapet **$p_{d_pptR} = h_{d_pptR} \times \gamma = 19.20 \text{ lb/ft}^2$**


SNOW LOAD - CANOPY
SNOW LOADING
In accordance with ASCE7-16

Tedds calculation version 1.0.12

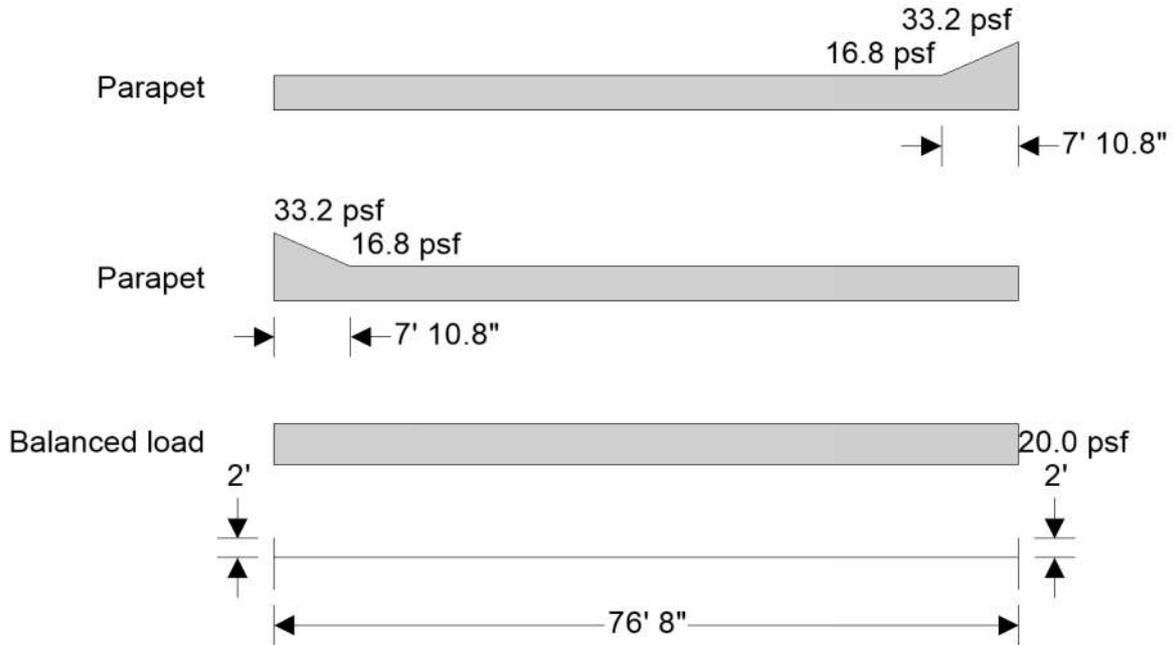
Building details

 Roof type Flat
 Width of roof $b = 76.67$ ft
Ground snow load

 Ground snow load (Figure 7.2-1) $p_g = 20.00$ lb/ft²
 Density of snow $\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = 16.60$ lb/ft³
 Surface roughness category (Sect. 26.7) C
 Exposure condition (Table 7.3-1) Partially exposed
 Exposure factor (Table 7.3-1) $C_e = 1.00$
 Thermal condition (Table 7.3-2) Unheated structures
 Thermal factor (Table 7.3-2) $C_t = 1.20$
 Importance category (Table 1.5-1) II
 Importance factor (Table 1.5-2) $I_s = 1.00$
 Min snow load for low slope roofs (Sect 7.3.4) $p_{f_min} = I_s \times p_g = 20.00$ lb/ft²
 Flat roof snow load (Sect 7.3) $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 16.80$ lb/ft²
Left parapet

 Balanced snow load height $h_b = p_f / \gamma = 1.01$ ft
 Height of left parapet $h_{pptL} = 2.00$ ft
 Height from balance load to top of left parapet $h_{c_pptL} = h_{pptL} - h_b = 0.99$ ft
 Length of roof - left parapet $l_{u_pptL} = b = 76.67$ ft
 Drift height windward drift - left parapet $h_{d_l_pptL} = \sqrt[3]{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10))^{1/4} - 1.5\text{ft}} = 2.08$ ft
 Drift height - left parapet $h_{d_pptL} = \min(h_{d_l_pptL}, h_{pptL} - h_b) = 0.99$ ft
 Drift width $W_{d_pptL} = \min(4 \times h_{d_l_pptL}^2 / h_{c_pptL}, 8 \times (h_{pptL} - h_b), b) = 7.90$ ft
 Drift surcharge load - left parapet $p_{d_pptL} = h_{d_pptL} \times \gamma = 16.40$ lb/ft²
Right parapet

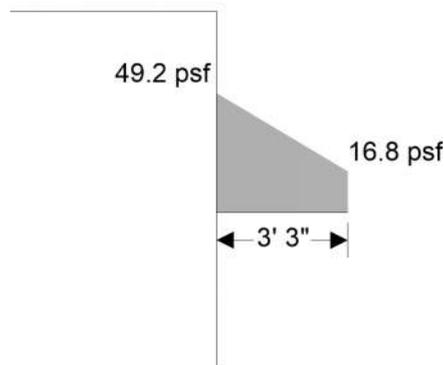
 Height of right parapet $h_{pptR} = 2.00$ ft
 Height from balance load to top of right parapet $h_{c_pptR} = h_{pptR} - h_b = 0.99$ ft
 Length of roof - right parapet $l_{u_pptR} = b = 76.67$ ft
 Drift height windward drift - right parapet $h_{d_l_pptR} = \sqrt[3]{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10))^{1/4} - 1.5\text{ft}} = 2.08$ ft
 Drift height - right parapet $h_{d_pptR} = \min(h_{d_l_pptR}, h_{pptR} - h_b) = 0.99$ ft
 Drift width $W_{d_pptR} = \min(4 \times h_{d_l_pptR}^2 / h_{c_pptR}, 8 \times (h_{pptR} - h_b), b) = 7.90$ ft
 Drift surcharge load - right parapet $p_{d_pptR} = h_{d_pptR} \times \gamma = 16.40$ lb/ft²



Roof elevation

Drift calculations

Balanced snow load height	$h_b = p_f / \gamma = 1.01 \text{ ft}$
Length of upper roof	$l_u = 76.67 \text{ ft}$
Length of lower roof	$l_l = 3.25 \text{ ft}$
Height diff between upper and lower roofs	$h_{diff} = 5.00 \text{ ft}$
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b = 3.99 \text{ ft}$
Drift height leeward drift	$h_{d_l} = \min(\sqrt{(l_s)} \times (0.43 \times (\max(20 \text{ ft}, l_u) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 10)^{1/4} - 1.5 \text{ ft}), 0.6 \times l_l) = 1.95 \text{ ft}$
Drift height windward drift	$h_{d_w} = \min(0.75 \times \sqrt{(l_s)} \times (0.43 \times (\max(20 \text{ ft}, l_l) \times 1 \text{ ft}^2)^{1/3} \times (p_g / 1 \text{ lb/ft}^2 + 10)^{1/4} - 1.5 \text{ ft}), \sqrt{(l_s \times p_g \times l_l / (4 \times \gamma))}) = 0.92 \text{ ft}$
Maximum lw/ww drift height	$h_{d_{max}} = \max(h_{d_w}, h_{d_l}) = 1.95 \text{ ft}$
Drift height	$h_d = \min(h_{d_{max}}, h_c) = 1.95 \text{ ft}$
Drift width	$W_d = \min(4 \times h_{d_{max}}, 8 \times h_c) = 7.80 \text{ ft}$
Drift surcharge load	$p_d = h_d \times \gamma = 32.37 \text{ lb/ft}^2$

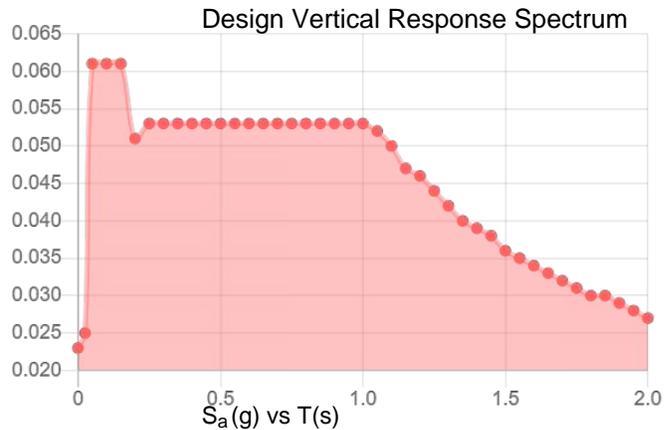
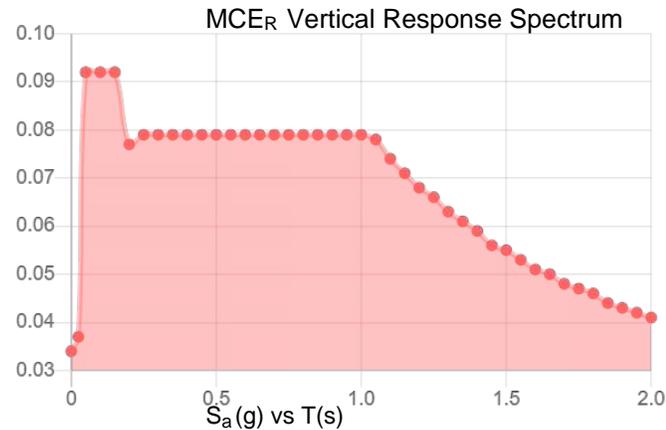
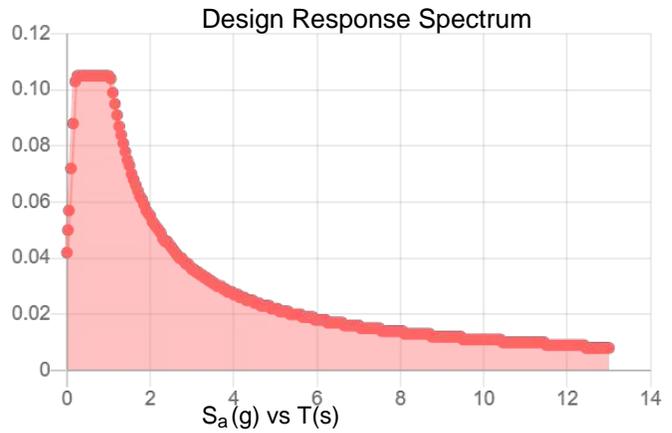
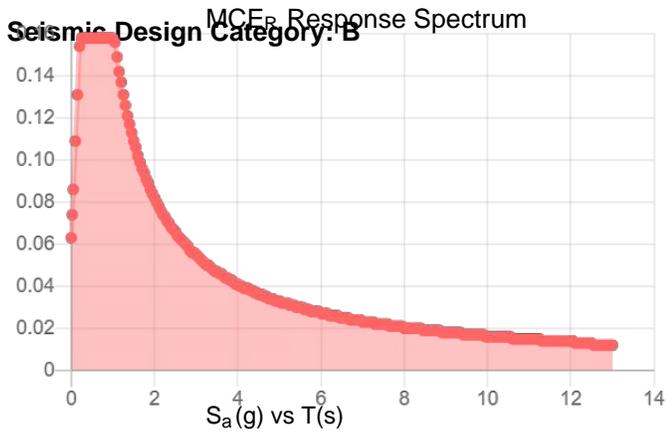


Elevation on snow drift

Site Soil Class: D - Stiff Soil

Results:

S_s :	0.099	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA _M :	0.075
S_{MS} :	0.158	F_{PGA} :	1.6
S_{M1} :	0.164	I_e :	1
S_{DS} :	0.105	C_v :	0.7



Data Accessed: Fri Jul 25 2025

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.05

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.2)	
at short period	$S_S = 0.099$
at 1 sec period	$S_1 = 0.068$
Site coefficient	
at short period (Table 11.4-1)	$F_a = 1.600$
at 1 sec period (Table 11.4-2)	$F_v = 2.400$

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S = 0.158$
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1 = 0.163$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS} = 0.106$
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1} = 0.109$

Seismic design category

Risk category (Table 1.5-1)	II
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Seismic design category based on short period response acceleration (Table 11.6-1)

A

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

B

Seismic design category

B

Approximate fundamental period

Height above base to highest level of building	$h_n = 13$ ft
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From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	$C_t = 0.02$
Building period parameter x	$x = 0.75$

 Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1 \text{sec} / (1 \text{ft})^x = 0.137$ sec

 Building fundamental period (Sect 12.8.2) $T = T_a = 0.137$ sec

 Long-period transition period $T_L = 12$ sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems 15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	$R = 6.5$
Seismic importance factor (Table 1.5-2)	$I_e = 1.000$
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s_calc} = S_{DS} / (R / I_e) = 0.0162$
Maximum (Eq 12.8-3)	$C_{s_max} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) = 0.1222$
Minimum (Eq.12.8-5)	$C_{s_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = 0.0100$
Seismic response coefficient	$C_s = 0.0162$

WOOD FRAMING

Typical Wall Studs

Wall Height to D.B.E. = 11.25 ft (average)
Parapet Height 1.75 ft (average)
Truss Depth = 22 in
Top Plate = 3 in
Bottom Plate = 1.5 in

Stud Height = 9.04 ft
Stud Spacing = 16 in
Building Length = 28.67 ft

Loading:

Roof Dead Load = 15 psf
Roof Live Load = 20 psf
Roof Snow (P_{fmin}) = 20 psf

Roof Snow Load (P_f) = 14 psf (when used with Drift)
Snow Drift = 33.2 psf
Snow Drift Length = 4.83 ft
Snow Drift at End = 46.37 plf

Wind Wall Pressure = 28.7 psf
(from C&C, Zone 5, Area 20sqft)

Wind Parapet Pressure = 52.9 psf
(from C&C, Zone 5p, Area 10sqft)

Wall Weight = 12 psf

Cumulative Axial Loads (lbs)

Dead Load = 494.7 lbs (roof + wall weight)
Roof Live Load = 382.3 lbs
Snow Load = 382.3 lbs (P_{f,min})
Snow Load = 329.4 lbs (drift)

Wind Load = 499.0 lb-ft

Load Combination:

	Axial	Moment
1.) DL + (LLr or SL)	877.0	N/A
2.) DL + 0.75(LLr or SL)	781.4	N/A
3.) DL +0.6WL	494.7	299.4
4.) D + 0.45WL + 0.75(LLr or SL)	781.4	224.6
5.) 0.6DL + 0.6WL	296.8	299.4

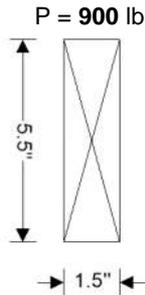
STRUCTURAL WOOD MEMBER DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design axial compression



Sawn lumber section details

Nominal breadth of sections	$b_{nom} = 2$ in
Dressed breadth of sections	$b = 1.5$ in
Nominal depth of sections	$d_{nom} = 6$ in
Dressed depth of sections	$d = 5.5$ in
Number of sections in member	$N = 1$
Overall breadth of member	$b_b = N \times b = 1.5$ in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = 900$ lb/in ²
Tension parallel to grain	$F_t = 575$ lb/in ²
Compression parallel to grain	$F_c = 1350$ lb/in ²
Compression perpendicular to grain	$F_{c_perp} = 625$ lb/in ²
Shear parallel to grain	$F_v = 180$ lb/in ²
Modulus of elasticity	$E = 1600000$ lb/in ²
Modulus of elasticity, stability calculations	$E_{min} = 580000$ lb/in ²
Mean shear modulus	$G_{def} = E / 16 = 100000$ lb/in ²

Member details

Service condition	Dry
Load duration	Ten years DL+LL
Unbraced length in x-axis	$L_x = 9.25$ ft
Effective length factor in x-axis	$K_x = 1$
Effective length in x-axis	$L_{ex} = L_x \times K_x = 9.25$ ft
Unbraced length in y-axis	$L_y = 1$ ft
Effective length factor in y-axis	$K_y = 1$
Effective length in y-axis	$L_{ey} = L_y \times K_y = 1$ ft

Section properties

Cross sectional area of member	$A = N \times b \times d = 8.25$ in ²
Section modulus	$S_x = N \times b \times d^2 / 6 = 7.56$ in ³
	$S_y = d \times (N \times b)^2 / 6 = 2.06$ in ³
Second moment of area	$I_x = N \times b \times d^3 / 12 = 20.80$ in ⁴
	$I_y = d \times (N \times b)^3 / 12 = 1.55$ in ⁴

Adjustment factors

Load duration factor - Table 2.3.2	$C_D = 1.00$
Temperature factor - Table 2.3.3	$C_t = 1.00$
Size factor for bending - Table 4A	$C_{Fb} = 1.30$
Size factor for tension - Table 4A	$C_{Ft} = 1.30$
Size factor for compression - Table 4A	$C_{Fc} = 1.10$
Flat use factor - Table 4A	$C_{fu} = 1.15$
Incising factor for modulus of elasticity - Table 4.3.8	$C_{iE} = 1.00$
Incising factor for bending, shear, tension & compression - Table 4.3.8	$C_i = 1.00$
Incising factor for perpendicular compression - Table 4.3.8	$C_{ic_perp} = 1.00$
Repetitive member factor - cl.4.3.9	$C_r = 1.00$
Bearing area factor - cl.3.10.4	$C_b = 1.00$
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} = 580000 \text{ lb/in}^2$
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i = 1485 \text{ lb/in}^2$
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = 1171 \text{ lb/in}^2$
	$c = 0.80$
Column stability factor - eq.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.60$
Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom}) = 3.00$
Effective laterally unsupported span length	$l_e = 9.25 \text{ ft}$
Slenderness ratio for bending members - eq.3.3-5	$R_b = \sqrt{[l_e \times d / (N \times b)]^2} = 16.472$
Adjusted bending design value for bending	$F_b^* = F_b \times C_D \times C_{Mb} \times C_t \times C_{Fb} \times C_i \times C_r = 1170 \text{ lb/in}^2$
Adjusted modulus of elasticity for member stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} = 580000 \text{ lb/in}^2$
Critical buckling design value for bending	$F_{bE} = 1.2 \times E_{min}' / R_b^2 = 2565 \text{ lb/in}^2$
Beam stability factor - eq.3.3-6	$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95} = 0.96$

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P = 897 \text{ lb/in}^2$
Applied compressive stress	$f_c = P / A = 109 \text{ lb/in}^2$
	$f_c / F_c' = 0.122$

PASS - Design compressive stress exceeds applied compressive stress

STRUCTURAL WOOD MEMBER DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

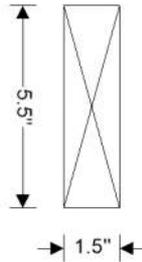
Analysis results

Design moment in major axis

$$M_x = 300 \text{ lb_ft}$$

Design axial compression

$$P = 495 \text{ lb}$$



Sawn lumber section details

Nominal breadth of sections

$$b_{\text{nom}} = 2 \text{ in}$$

Dressed breadth of sections

$$b = 1.5 \text{ in}$$

Nominal depth of sections

$$d_{\text{nom}} = 6 \text{ in}$$

Dressed depth of sections

$$d = 5.5 \text{ in}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 1.5 \text{ in}$$

Species, grade and size classification

Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain

$$F_b = 900 \text{ lb/in}^2$$

Tension parallel to grain

$$F_t = 575 \text{ lb/in}^2$$

Compression parallel to grain

$$F_c = 1350 \text{ lb/in}^2$$

Compression perpendicular to grain

$$F_{c_perp} = 625 \text{ lb/in}^2$$

Shear parallel to grain

$$F_v = 180 \text{ lb/in}^2$$

Modulus of elasticity

$$E = 1600000 \text{ lb/in}^2$$

Modulus of elasticity, stability calculations

$$E_{\text{min}} = 580000 \text{ lb/in}^2$$

Mean shear modulus

$$G_{\text{def}} = E / 16 = 100000 \text{ lb/in}^2$$

Member details

Service condition

Dry

Load duration

Ten minutes WIND

Unbraced length in x-axis

$$L_x = 9.25 \text{ ft}$$

Effective length factor in x-axis

$$K_x = 1$$

Effective length in x-axis

$$L_{ex} = L_x \times K_x = 9.25 \text{ ft}$$

Unbraced length in y-axis

$$L_y = 1 \text{ ft}$$

Effective length factor in y-axis

$$K_y = 1$$

Effective length in y-axis

$$L_{ey} = L_y \times K_y = 1 \text{ ft}$$

Section properties

Cross sectional area of member

$$A = N \times b \times d = 8.25 \text{ in}^2$$

Section modulus

$$S_x = N \times b \times d^2 / 6 = 7.56 \text{ in}^3$$

$$S_y = d \times (N \times b)^2 / 6 = 2.06 \text{ in}^3$$

Second moment of area

$$I_x = N \times b \times d^3 / 12 = 20.80 \text{ in}^4$$

$$I_y = d \times (N \times b)^3 / 12 = 1.55 \text{ in}^4$$

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = 1.60$

Temperature factor - Table 2.3.3 $C_t = 1.00$

Size factor for bending - Table 4A $C_{Fb} = 1.30$

Size factor for tension - Table 4A $C_{Ft} = 1.30$

Size factor for compression - Table 4A $C_{Fc} = 1.10$

Flat use factor - Table 4A $C_{fu} = 1.15$

Incising factor for modulus of elasticity - Table 4.3.8
 $C_{iE} = 1.00$

Incising factor for bending, shear, tension & compression - Table 4.3.8
 $C_i = 1.00$

Incising factor for perpendicular compression - Table 4.3.8
 $C_{ic_perp} = 1.00$

Repetitive member factor - cl.4.3.9 $C_r = 1.00$

Bearing area factor - cl.3.10.4 $C_b = 1.00$

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} = 580000 \text{ lb/in}^2$

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i = 2376 \text{ lb/in}^2$

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = 1171 \text{ lb/in}^2$
 $c = 0.80$

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.43$$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = 3.00$

Effective laterally unsupported span length $l_e = 9.25 \text{ ft}$

Slenderness ratio for bending members - eq.3.3-5 $R_b = \sqrt{[l_e \times d / (N \times b)]^2} = 16.472$

Adjusted bending design value for bending $F_b^* = F_b \times C_D \times C_{Mb} \times C_t \times C_{Fb} \times C_i \times C_r = 1872 \text{ lb/in}^2$

Adjusted modulus of elasticity for member stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE} = 580000 \text{ lb/in}^2$

Critical buckling design value for bending $F_{bE} = 1.2 \times E_{min}' / R_b^2 = 2565 \text{ lb/in}^2$

Beam stability factor - eq.3.3-6

$$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[(1 + (F_{bE} / F_b^*)) / 1.9]^2 - (F_{bE} / F_b^*) / 0.95} = 0.91$$

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = 1704 \text{ lb/in}^2$

Actual bending stress $f_b = M_x / S_x = 476 \text{ lb/in}^2$

$$f_b / F_b' = 0.279$$

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P = 1018 \text{ lb/in}^2$

Applied compressive stress $f_c = P / A = 60 \text{ lb/in}^2$

$$f_c / F_c' = 0.059$$

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = 1171 \text{ lb/in}^2$

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})]) = 0.298 < 1$

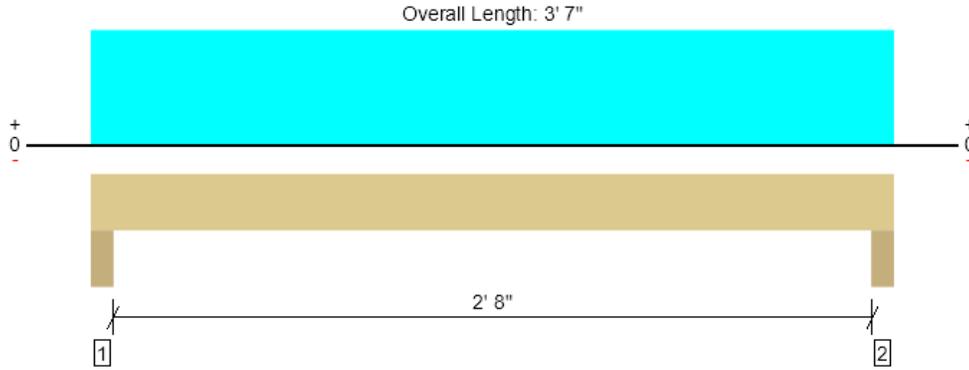
PASS - Combined compressive and bending stresses are within permissible limits

Level			
Member Name	Results (Max UTIL %)	Current Solution	Comments
H1 - East Side - Typ. Door Header	Passed (6% B/C)	1 piece(s) 6 x 6 DF No.2	
H2 South Entry Door - 6'4"	Passed (73% M)	1 piece(s) 6 x 8 DF No.2	
H3 South Side - Window 10ft	Passed (51% ΔT)	1 piece(s) 5 1/4" x 9 1/2" 2.0E Parallam® PSL	
H4 West Side Window 10ft - (overhead Door)	Passed (61% M)	1 piece(s) 6 x 10 DF No.2	

ForteWEB Software Operator	Job Notes
Carmen Fernandez BSE Structural Engineers (405) 482-7215 cfernandez@bsestructural.com	



Level, H1 - East Side - Typ. Door Header
1 piece(s) 6 x 6 DF No.2



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	186 @ 4"	18906 (5.50")	Passed (1%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	91 @ 11"	3943	Passed (2%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	111 @ 1' 9 1/2"	1993	Passed (6%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.001 @ 1' 9 1/2"	0.097	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.002 @ 1' 9 1/2"	0.146	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	138 @ 3' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	83 @ 11"	5485	Passed (2%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	101 @ mid-span	2773	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.001 @ mid-span	0.097	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.06	1.00	Passed (6%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

Member Length : 3' 7"
 System : Wall
 Member Type : Header
 Building Use : Residential
 Building Code : IBC 2018
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.86 to account for the beam stability and/or volume/size factors.
- Applicable calculations are based on NDS.
- This product has a square cross section. The analysis engine has checked both edge and plank orientations to allow for either installation.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Plate - DF	5.50"	5.50"	1.50"	67	72	119	186	None
2 - Plate - DF	5.50"	5.50"	1.50"	67	72	119	186	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	2	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	2	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 7"	N/A	7.7	--	--	
1 - Uniform (PSF)	0 to 3' 7"	2'	15.0	20.0	33.2	Typical Roof Loading

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	5' 6"	28.7	C&C - Zone 5, 20sqft

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

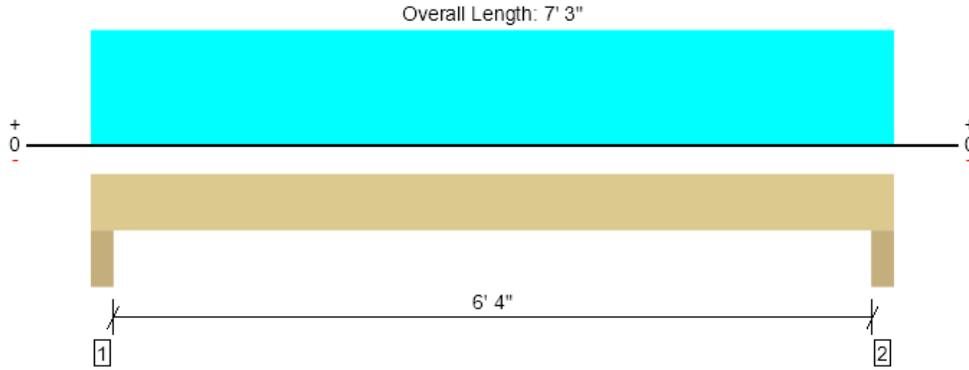
Forteweb Software Operator	Job Notes
Carmen Fernandez BSE Structural Engineers (405) 482-7215 cfernandez@bsestructural.com	



11/20/2025 2:03:23 PM UTC
 Forteweb v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
 File Name: 25-290 Paragon Star - Headers

Level, H2 South Entry Door - 6'4"

1 piece(s) 6 x 8 DF No.2



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1793 @ 4"	18906 (5.50")	Passed (9%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1257 @ 1' 1"	5376	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2680 @ 3' 7 1/2"	3695	Passed (73%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.047 @ 3' 7 1/2"	0.219	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.083 @ 3' 7 1/2"	0.329	Passed (L/950)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	255 @ 6' 11"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	210 @ 11"	7480	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	420 @ mid-span	3781	Passed (11%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.017 @ mid-span	0.219	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.53	1.00	Passed (53%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

Member Length : 7' 3"
 System : Wall
 Member Type : Header
 Building Use : Residential
 Building Code : IBC 2018
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.85 to account for the beam stability and/or volume/size factors.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Plate - DF	5.50"	5.50"	1.50"	790	1003	1003	1793	None
2 - Plate - DF	5.50"	5.50"	1.50"	790	1003	1003	1793	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 3"	N/A	10.4	--	--	
1 - Uniform (PSF)	0 to 7' 3"	13' 10"	15.0	20.0	20.0	Typical Roof Loading

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	28.7	C&C - Zone 5, 20sqft

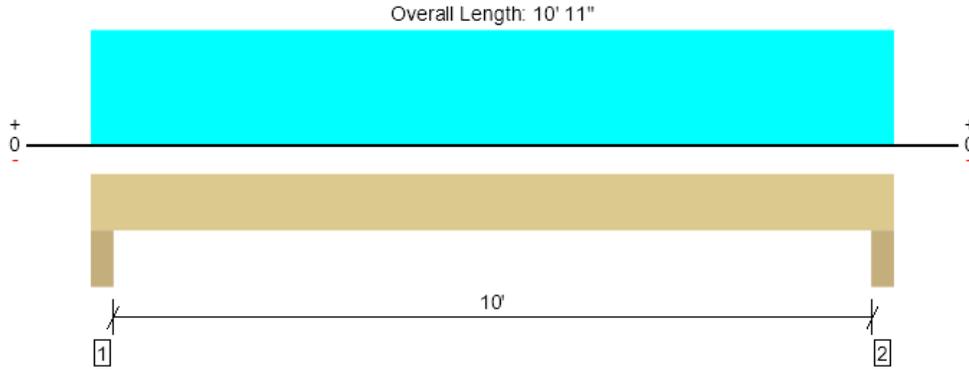
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Forteweb Software Operator	Job Notes
Carmen Fernandez BSE Structural Engineers (405) 482-7215 cfernandez@bsestructural.com	



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 File Name: 25-290 Paragon Star - Headers

Level, H3 South Side - Window 10ft
1 piece(s) 5 1/4" x 9 1/2" 2.0E Parallam® PSL



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3984 @ 4"	18047 (5.50")	Passed (22%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3071 @ 1' 3"	11089	Passed (28%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	9585 @ 5' 5 1/2"	22213	Passed (43%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.163 @ 5' 5 1/2"	0.342	Passed (L/755)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.264 @ 5' 5 1/2"	0.512	Passed (L/466)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	265 @ 10' 7"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	236 @ 10 3/4"	11172	Passed (2%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	678 @ mid-span	16902	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.040 @ mid-span	0.342	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.29	1.00	Passed (29%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

Member Length : 10' 11"
 System : Wall
 Member Type : Header
 Building Use : Residential
 Building Code : IBC 2018
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Initial eccentricity applied as per ESR-1387.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Plate - DF	5.50"	5.50"	1.50"	1524	1919	2459	3984	None
2 - Plate - DF	5.50"	5.50"	1.50"	1524	1919	2459	3984	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 10' 11"	N/A	15.6	--	--	
1 - Uniform (PSF)	0 to 10' 11"	14' 3 15/16"	15.0	20.0	14.0	Typical Roof Loading
2 - Uniform (PLF)	0 to 10' 11"	N/A	--	--	90.0	Snow Drift
3 - Uniform (PSF)	0 to 10' 11"	3' 3"	15.0	20.0	49.2	Canopy

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	3'	28.7	C&C - Zone 5, 20sqft

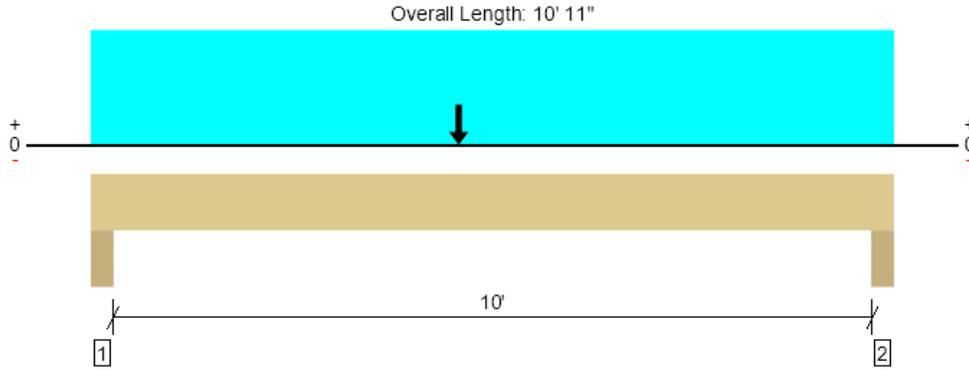
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Carmen Fernandez BSE Structural Engineers (405) 482-7215 cfernandez@bsestructural.com	



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 File Name: 25-290 Paragon Star - Headers

Level, H4 West Side Window 10ft -(overhead Door)
1 piece(s) 6 x 10 DF No.2



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1737 @ 4"	18906 (5.50")	Passed (9%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1339 @ 1' 3"	6810	Passed (20%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4180 @ 5' 5 1/2"	6887	Passed (61%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.110 @ 5' 5 9/16"	0.342	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.155 @ 5' 5 9/16"	0.512	Passed (L/795)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	353 @ 10' 7"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	313 @ 11"	9475	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	905 @ mid-span	5588	Passed (16%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.070 @ mid-span	0.342	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.48	1.00	Passed (48%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

Member Length : 10' 11"
 System : Wall
 Member Type : Header
 Building Use : Residential
 Building Code : IBC 2018
 Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.99 to account for the beam stability and/or volume/size factors.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Plate - DF	5.50"	5.50"	1.50"	502	682	1235	1737	None
2 - Plate - DF	5.50"	5.50"	1.50"	502	664	1235	1737	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 10' 11"	N/A	13.2	--	--	
1 - Uniform (PSF)	0 to 10' 11"	2'	15.0	20.0	33.2	Typical Roof Loading
2 - Point (lb)	5'	N/A	--	200	--	Point Load for Overhead Door
3 - Uniform (PSF)	0 to 10' 11"	3' 3"	15.0	20.0	49.2	Canopy Loads

Forteweb Software Operator	Job Notes
Carmen Fernandez BSE Structural Engineers (405) 482-7215 cfernandez@bsestructural.com	



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 File Name: 25-290 Paragon Star - Headers

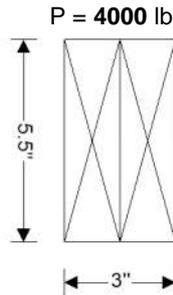
**HEADER BEARING STUDS
H3 - Controlling**
STRUCTURAL WOOD MEMBER DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design axial compression


Sawn lumber section details

Nominal breadth of sections	$b_{nom} = 2$ in
Dressed breadth of sections	$b = 1.5$ in
Nominal depth of sections	$d_{nom} = 6$ in
Dressed depth of sections	$d = 5.5$ in
Number of sections in member	$N = 2$
Overall breadth of member	$b_b = N \times b = 3$ in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = 900$ lb/in ²
Tension parallel to grain	$F_t = 575$ lb/in ²
Compression parallel to grain	$F_c = 1350$ lb/in ²
Compression perpendicular to grain	$F_{c_perp} = 625$ lb/in ²
Shear parallel to grain	$F_v = 180$ lb/in ²
Modulus of elasticity	$E = 1600000$ lb/in ²
Modulus of elasticity, stability calculations	$E_{min} = 580000$ lb/in ²
Mean shear modulus	$G_{def} = E / 16 = 100000$ lb/in ²

Member details

Service condition	Dry
Load duration	Ten years
Unbraced length in x-axis	$L_x = 8$ ft
Effective length factor in x-axis	$K_x = 1$
Effective length in x-axis	$L_{ex} = L_x \times K_x = 8$ ft
Unbraced length in y-axis	$L_y = 1$ ft
Effective length factor in y-axis	$K_y = 1$
Effective length in y-axis	$L_{ey} = L_y \times K_y = 1$ ft

Section properties

Cross sectional area of member	$A = N \times b \times d = 16.50$ in ²
Section modulus	$S_x = N \times b \times d^2 / 6 = 15.12$ in ³
	$S_y = d \times (N \times b)^2 / 6 = 8.25$ in ³
Second moment of area	$I_x = N \times b \times d^3 / 12 = 41.59$ in ⁴
	$I_y = d \times (N \times b)^3 / 12 = 12.37$ in ⁴

Adjustment factors

Load duration factor - Table 2.3.2	$C_D = 1.00$
Temperature factor - Table 2.3.3	$C_t = 1.00$
Size factor for bending - Table 4A	$C_{Fb} = 1.30$
Size factor for tension - Table 4A	$C_{Ft} = 1.30$
Size factor for compression - Table 4A	$C_{Fc} = 1.10$
Flat use factor - Table 4A	$C_{fu} = 1.15$
Incising factor for modulus of elasticity - Table 4.3.8	$C_{IE} = 1.00$
Incising factor for bending, shear, tension & compression - Table 4.3.8	$C_i = 1.00$
Incising factor for perpendicular compression - Table 4.3.8	$C_{ic_perp} = 1.00$
Repetitive member factor - cl.4.3.9	$C_r = 1.00$
Bearing area factor - cl.3.10.4	$C_b = 1.00$
Column stability coefficient – cl.15.3.2	$K_f = 0.60$
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{IE} = 580000 \text{ lb/in}^2$
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i = 1485 \text{ lb/in}^2$
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b_b)^2 = 29797 \text{ lb/in}^2$
	$c = 0.80$
Column stability factor - eq.15.3-1	$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c)] - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.59$
Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom}) = 1.50$
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	$C_L = 1.00$
Strength in compression parallel to grain - cl.3.6.3	
Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P = 882 \text{ lb/in}^2$
Applied compressive stress	$f_c = P / A = 242 \text{ lb/in}^2$
	$f_c / F_c' = 0.275$
PASS - Design compressive stress exceeds applied compressive stress	

STRUCTURAL WOOD MEMBER DESIGN (NDS)

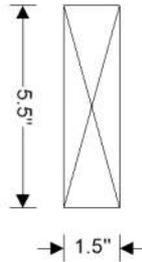
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis
Design shear
Design axial compression

$M_x = 872 \text{ lb_ft}$
 $F = 388 \text{ lb}$
 $P = 670 \text{ lb}$



Wind Load = 28.7psf (C&C Zone 5, 20sqft)
Header Span = 10ft
King Stud Height = 9ft approx.

$M = WL^2 / 8 = (28.7\text{psf} * 0.6\text{asd}) * (10/2) * (9^2) / 8 = 872\text{lb-ft}$

$V = WL/2 = (28.7\text{psf} * 0.6\text{asd}) * (10/2) * (9) / 2 = 388\text{lb-ft}$

$P = (15\text{psf} + 20\text{psf}) * (28.67/2) * (16/12) = 670\text{lbs}$

Sawn lumber section details

Nominal breadth of sections
Dressed breadth of sections
Nominal depth of sections
Dressed depth of sections
Number of sections in member
Overall breadth of member
Species, grade and size classification
Bending parallel to grain
Tension parallel to grain
Compression parallel to grain
Compression perpendicular to grain
Shear parallel to grain
Modulus of elasticity
Modulus of elasticity, stability calculations
Mean shear modulus

$b_{\text{nom}} = 2 \text{ in}$
 $b = 1.5 \text{ in}$
 $d_{\text{nom}} = 6 \text{ in}$
 $d = 5.5 \text{ in}$
 $N = 1$
 $b_b = N \times b = 1.5 \text{ in}$
Douglas Fir-Larch, No.2 grade, 2" & wider
 $F_b = 900 \text{ lb/in}^2$
 $F_t = 575 \text{ lb/in}^2$
 $F_c = 1350 \text{ lb/in}^2$
 $F_{c_perp} = 625 \text{ lb/in}^2$
 $F_v = 180 \text{ lb/in}^2$
 $E = 1600000 \text{ lb/in}^2$
 $E_{\text{min}} = 580000 \text{ lb/in}^2$
 $G_{\text{def}} = E / 16 = 100000 \text{ lb/in}^2$

Member details

Service condition
Load duration
Unbraced length in x-axis
Effective length factor in x-axis
Effective length in x-axis
Unbraced length in y-axis
Effective length factor in y-axis
Effective length in y-axis

Dry
Ten minutes
 $L_x = 9 \text{ ft}$
 $K_x = 1$
 $L_{\text{ex}} = L_x \times K_x = 9 \text{ ft}$
 $L_y = 1 \text{ ft}$
 $K_y = 1$
 $L_{\text{ey}} = L_y \times K_y = 1 \text{ ft}$

Section properties

Cross sectional area of member
Section modulus
Second moment of area

$A = N \times b \times d = 8.25 \text{ in}^2$
 $S_x = N \times b \times d^2 / 6 = 7.56 \text{ in}^3$
 $S_y = d \times (N \times b)^2 / 6 = 2.06 \text{ in}^3$
 $I_x = N \times b \times d^3 / 12 = 20.80 \text{ in}^4$

$$I_y = d \times (N \times b)^3 / 12 = \mathbf{1.55 \text{ in}^4}$$

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.60}$

Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$

Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.30}$

Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.30}$

Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.10}$

Flat use factor - Table 4A $C_{fu} = \mathbf{1.15}$

Incising factor for modulus of elasticity - Table 4.3.8 $C_{IE} = \mathbf{1.00}$

Incising factor for bending, shear, tension & compression - Table 4.3.8 $C_i = \mathbf{1.00}$

Incising factor for perpendicular compression - Table 4.3.8 $C_{ic_perp} = \mathbf{1.00}$

Repetitive member factor - cl.4.3.9 $C_r = \mathbf{1.00}$

Bearing area factor - cl.3.10.4 $C_b = \mathbf{1.00}$

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{IE} = \mathbf{580000 \text{ lb/in}^2}$

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i = \mathbf{2376 \text{ lb/in}^2}$

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = \mathbf{1236 \text{ lb/in}^2}$
 $c = \mathbf{0.80}$

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = \mathbf{0.45}$$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom}) = \mathbf{3.00}$

- Beam is fully restrained

Beam stability factor - cl.3.3.3 $C_L = \mathbf{1.00}$

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1872 \text{ lb/in}^2}$

Actual bending stress $f_b = M_x / S_x = \mathbf{1384 \text{ lb/in}^2}$

$$f_b / F_b' = \mathbf{0.739}$$

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P = \mathbf{1064 \text{ lb/in}^2}$

Applied compressive stress $f_c = P / A = \mathbf{81 \text{ lb/in}^2}$

$$f_c / F_c' = \mathbf{0.076}$$

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = \mathbf{1236 \text{ lb/in}^2}$

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})]) = \mathbf{0.797} < \mathbf{1}$

PASS - Combined compressive and bending stresses are within permissible limits

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{288 \text{ lb/in}^2}$

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{71 \text{ lb/in}^2}$

$$f_v / F_v' = \mathbf{0.245}$$

PASS - Design shear stress exceeds actual shear stress

FOUNDATIONS

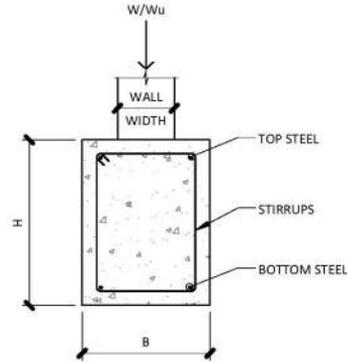
Grade Beam Design

Grade Beam Location: Typical

General Information:

Footing Width, B = **18.00 in.**
 Footing Depth, H = **34.00 in.**
 Steel Depth, d = **30.25 in.**
 Wall Width = **5.50 in.**
 Soil Bearing Pressure = **1.50 ksf**
 Allowable or Effective SBC? **Effective**
 Footing Concrete Strength = **3.00 ksi**
 Wall Concrete Strength = **3.00 ksi**

(H - 3 in - 1.5*Bar Dia.)



Loading:

Vertical Loads:
 Applied Dead Load = **0.215 klf**
 Wall Weight = **12.00 psf**
 Wall Height = **13.00 ft.**
 Total Wall Weight = 0.16 klf
 Footing Weight = 0.64 klf
 Applied Live Load = **0.287 klf**
 ASD Total Load, W = 0.66 klf
 LRFD Total Load, Wu = 0.90 klf

LRFD Factors: (ASCE 7 Combo)
 Dead = **1.2**
 Live = **1.6**

ASD Soil Pressures:

Required Footing Width = 0.44 ft.
 Qmax = 438.73 psf
 Chosen Footing Width = 1.50 ft.
 Assumed Footing Span = **6.00 ft.**
 Is Qmax < SBC? **YES**

(Footing Width = W / Soil Bearing Pressure)
 (Actual Soil Bearing Pressure = W / Footing Width)

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure = 603.01 psf
 h = 32.00 in.
 Cantilever = 6.25 in.
 Vu1 = 0.31 klf
 ϕV_n = 21.03 klf

(h = H - 2 in.)
 (Cantilever = B/2 - Wall Width/2)
 (LRFD Bearing Pressure* Cantilever)
 (ACI 318-14 Equation 14.5.5.1, $\phi V_n = 0.75*(4/3)*\sqrt{f_c}$

Adequate in One-Way Shear? **YES**

Plain Concrete Flexure Check: Cantilevered Side of Footing

h = 32.00 in.
 Cantilever = 6.25 in.
 Mu = 0.08 k-ft/ft
 S = 3072.00 cu. in.
 ϕM_n = 63.10 k-ft/ft

(h = H - 2 in.)
 (Cantilever = B/2 - Wall Width/2)
 (Actual Soil Bearing Pressure* Cantilever)
 (S = $12*h^2 / 4$)
 (ACI 318-14 Equation 14.5.2.1 (a) & (b), $\phi M_n = \min c$
 and $0.9*0.85*(f_c)*S / 1000*12$)

Adequate in Flexure? **YES**

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 = 2.71 klf
 ϕV_n = 44.74 klf

(Wu*Assumed Footing Span / 2)
 (ACI 318-14 Equation 22.5.5.1, $\phi V_n = 0.75*2*\sqrt{f_c}$

Adequate in One-Way Shear? **YES**

Are Stirrups Req'd? **NO**

Use #3 Stirrups @ 18 in. O.C.

(ACI 318-14 Section 7.6.3.1 If $\phi V_n/2 > Vu1$ "No", Other
 (Provide minimum stirrups to support steel)

Wall Bearing Check:

$\phi P_n = 109.40$ klf (ACI 318-14 Table 22.8.3.2 $\phi P_n = 0.65 \cdot 0.85 \cdot f_c \cdot \text{Plate}$)
 Adequate in Bearing? **YES**

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

$M_u = 4.07$ k-ft ($W_u \cdot \text{Assumed Footing Span}^2 / 8$)
 $m = 23.529$ ($m = f_y / (0.85 \cdot f_c)$)
 $R_u = 0.00$ ksi ($R_u = M_u / (0.9 \cdot B \cdot d^2)$)
 $\rho \text{ Req'd} = 0.0001$ ($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot R_u \cdot m / f_y})$)
 $\rho \text{ Min.} = 0.0027$ (ACI 318-14 Equations 8.7.5.6.3.1 (a) and (b), Smaller)
 $4/3 \cdot \mu \rho \text{ Req'd} = 0.0001$ ($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot 1.33 \cdot R_u \cdot m / f_y})$)
 Governing $\rho = 0.0001$ (If $\rho \text{ Req'd} < 4/3 \cdot \mu \rho \text{ Req'd} < \rho \text{ Min.}$, Use $4/3 \cdot \mu \rho$)
 $A_s \text{ Required} = 0.04$ sq.in. ($A_s = \text{Governing } \rho \cdot B \cdot d$)
 $\text{Bar \#} = 4$
 $\text{Number of Bars} = 3$ bars
 $A_s \text{ Provided} = 0.60$ sq.in.
 Adequate Bott. Reinf. Provided? **YES**

Top Reinforcement:

$A_s \text{ Required} = 0.00$ sq.in.
 $\text{Bar \#} = 4$
 $\text{Number of Bars} = 3$ bars
 $A_s \text{ Provided} = 0.60$ sq.in.
 Adequate Bott. Reinf. Provided? **YES**

Temperature & Shrinkage Reinforcement:

Minimum Steel = 1.10 sq.in. (ACI 318-14 Table 24.4.3.2 T&S Reinf. = $0.0018 \cdot 12$ in)
 $A_s \text{ Provided Top} = 0.60$ sq.in.
 $A_s \text{ Provided Bott} = 0.60$ sq.in.
 $A_s \text{ Provided Total} = 1.20$ sq.in.
 T&S Steel Provided? **YES**

Design Checks:

Is $Q_{max} < SBC$? **YES**
 Adequate in One-Way Shear? **YES**
 Adequate in One-Way Shear (Span)? **YES**
 Adequate in Flexure? **YES**
 Adequate in Flexure (Span)? **YES**
 Adequate in Bearing? **YES**
 Adequate Top Reinf. Provided? **YES**
 Adequate Bott. Reinf. Provided? **YES**
 T&S Reinf. Provided? **YES**

Footing Design:

Footing Width, $B = 18.00$ in.
 Footing Depth, $H = 34.00$ in.
 $\text{Top Steel} = (3) \#4$ bars
 $\text{Bottom Steel} = (3) \#4$ bars
 $\text{Stirrups} = \#3 \text{ Stirrups @ } 18 \text{ in. O.C.}$

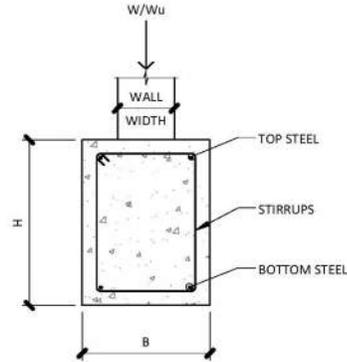
Grade Beam Design

Grade Beam Location: Double Bathroom Chase Wall
Thickened Slab

General Information:

Footing Width, B = **24.00 in.**
 Footing Depth, H = **12.00 in.**
 Steel Depth, d = **8.25 in.**
 Wall Width = **12.50 in.**
 Soil Bearing Pressure = **1.50 ksf**
 Allowable or Effective SBC? **Effective**
 Footing Concrete Strength = **3.00 ksi**
 Wall Concrete Strength = **3.00 ksi**

(H - 3 in - 1.5*Bar Dia.)



Loading:

Vertical Loads:
 Applied Dead Load = **0.030 klf**
 Wall Weight = **10.00 psf**
 Wall Height = **11.00 ft.**
 Total Wall Weight = **0.11 klf**
 Footing Weight = **0.30 klf**
 Applied Live Load = **0.040 klf**
 ASD Total Load, W = **0.18 klf**
 LRFD Total Load, Wu = **0.23 klf**

LRFD Factors: (ASCE 7 Combo)
 Dead = **1.2**
 Live = **1.6**

ASD Soil Pressures:

Required Footing Width = **0.12 ft.**
 $Q_{max} = 90.00 \text{ psf}$
 Chosen Footing Width = **2.00 ft.**
 Assumed Footing Span = **6.00 ft.**
 Is $Q_{max} < SBC$? **YES**

(Footing Width = W / Soil Bearing Pressure)
 (Actual Soil Bearing Pressure = W / Footing Width)

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure = **116.00 psf**
 $h = 10.00 \text{ in.}$
 Cantilever = **5.75 in.**
 $V_{u1} = 0.06 \text{ klf}$
 $\phi V_n = 6.57 \text{ klf}$

($h = H - 2 \text{ in.}$)
 (Cantilever = $B/2 - \text{Wall Width}/2$)
 (LRFD Bearing Pressure* Cantilever)
 (ACI 318-14 Equation 14.5.5.1, $\phi V_n = 0.75*(4/3)*\text{sqrt}$

Adequate in One-Way Shear? **YES**

Plain Concrete Flexure Check: Cantilevered Side of Footing

$h = 10.00 \text{ in.}$
 Cantilever = **5.75 in.**
 $M_u = 0.01 \text{ k-ft/ft}$
 $S = 300.00 \text{ cu. in.}$
 $\phi M_n = 6.16 \text{ k-ft/ft}$

($h = H - 2 \text{ in.}$)
 (Cantilever = $B/2 - \text{Wall Width}/2$)
 (Actual Soil Bearing Pressure* Cantilever)
 ($S = 12*h^2 / 4$)
 (ACI 318-14 Equation 14.5.2.1 (a) & (b), $\phi M_n = \min c$
 and $0.9*0.85*(f_c)*S / 1000*12$)

Adequate in Flexure? **YES**

One-Way Shear Check: For Spanning "X" Distance Listed

$V_{u1} = 0.70 \text{ klf}$
 $\phi V_n = 16.27 \text{ klf}$

($W_u*\text{Assumed Footing Span} / 2$)
 (ACI 318-14 Equation 22.5.5.1, $\phi V_n = 0.75*2*\text{sqrt}(f_c)$

Adequate in One-Way Shear? **YES**

Are Stirrups Req'd? **NO**

Use #3 Stirrups @ 18 in. O.C.

(ACI 318-14 Section 7.6.3.1 If $\phi V_n/2 > V_{u1}$ "No", Other
 (Provide minimum stirrups to support steel)

Wall Bearing Check:

$\Phi P_n = 248.63$ klf (ACI 318-14 Table 22.8.3.2 $\Phi P_n = 0.65 \cdot 0.85 \cdot f_c \cdot \text{Plate}$)
 Adequate in Bearing? **YES**

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	1.04 k-ft	(Wu*Assumed Footing Span ² / 8)
m =	23.529	(m = fy/(0.85*fc))
Ru =	0.01 ksi	(Ru = Mu/(0.9*B*d ²))
ρ Req'd =	0.0001	($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot Ru \cdot m / fy})$)
ρ Min. =	0.0027	(ACI 318-14 Equations 8.7.5.6.3.1 (a) and (b), Smaller
4/3*Mu ρ Req'd =	0.0002	($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot 1.33 \cdot Ru \cdot m / fy})$)
Governing ρ =	0.0002	(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ F
A's Required =	0.04 sq.in.	(As = Governing $\rho \cdot B \cdot d$)
Bar # =	4	
Number of Bars =	3 bars	
As Provided =	0.60 sq.in.	
Adequate Bott. Reinf. Provided?	YES	

Top Reinforcement:

A's Required = 0.00 sq.in.
 Bar # = 4
 Number of Bars = 0 bars
 As Provided = 0.00 sq.in.
 Adequate Bott. Reinf. Provided? **YES**

Temperature & Shrinkage Reinforcement:

Minimum Steel = 0.52 sq.in. (ACI 318-14 Table 24.4.3.2 T&S Reinf. = 0.0018*12 in)
 As Provided Top = 0.00 sq.in.
 As Provided Bott = 0.60 sq.in.
 As Provided Total = 0.60 sq.in.
 T&S Steel Provided? **YES**

Design Checks:

Is Qmax < SBC? **YES**
 Adequate in One-Way Shear? **YES**
 Adequate in One-Way Shear (Span)? **YES**
 Adequate in Flexure? **YES**
 Adequate in Flexure (Span)? **YES**
 Adequate in Bearing? **YES**
 Adequate Top Reinf. Provided? **YES**
 Adequate Bott. Reinf. Provided? **YES**
 T&S Reinf. Provided? **YES**

Footing Design:	
Footing Width, B =	24.00 in.
Footing Depth, H =	12.00 in.
Top Steel =	() #4 bars
Bottom Steel =	(3) #4 bars
Stirrups =	#3 Stirrups @ 18 in. O.C.

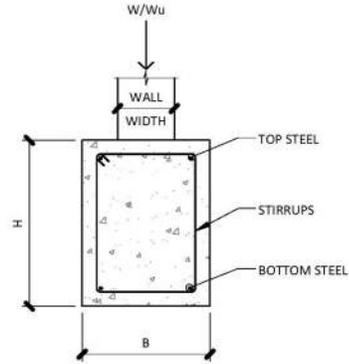
Grade Beam Design

Grade Beam Location: Typ. Thickened Slab

General Information:

Footing Width, B = **18.00 in.**
 Footing Depth, H = **12.00 in.**
 Steel Depth, d = **8.25 in.**
 Wall Width = **3.50 in.**
 Soil Bearing Pressure = **1.50 ksf**
 Allowable or Effective SBC? **Effective**
 Footing Concrete Strength = **3.00 ksi**
 Wall Concrete Strength = **3.00 ksi**

(H - 3 in - 1.5*Bar Dia.)



Loading:

Vertical Loads:
 Applied Dead Load = **0.030 klf**
 Wall Weight = **10.00 psf**
 Wall Height = **11.00 ft.**
 Total Wall Weight = **0.11 klf**
 Footing Weight = **0.23 klf**
 Applied Live Load = **0.040 klf**
 ASD Total Load, W = **0.18 klf**
 LRFD Total Load, Wu = **0.23 klf**

LRFD Factors: (ASCE 7 Combo)
 Dead = **1.2**
 Live = **1.6**

ASD Soil Pressures:

Required Footing Width = **0.12 ft.**
 Qmax = **120.00 psf**
 Chosen Footing Width = **1.50 ft.**
 Assumed Footing Span = **6.00 ft.**
 Is Qmax < SBC? **YES**

(Footing Width = W / Soil Bearing Pressure)
 (Actual Soil Bearing Pressure = W / Footing Width)

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure = **154.67 psf**
 h = **10.00 in.**
 Cantilever = **7.25 in.**
 Vu1 = **0.09 klf**
 ΦVn = **6.57 klf**

(h = H - 2 in.)
 (Cantilever = B/2 - Wall Width/2)
 (LRFD Bearing Pressure* Cantilever)
 (ACI 318-14 Equation 14.5.5.1, ΦVn = 0.75*(4/3)*sqrt

Adequate in One-Way Shear? **YES**

Plain Concrete Flexure Check: Cantilevered Side of Footing

h = **10.00 in.**
 Cantilever = **7.25 in.**
 Mu = **0.03 k-ft/ft**
 S = **300.00 cu. in.**
 ΦMn = **6.16 k-ft/ft**

(h = H - 2 in.)
 (Cantilever = B/2 - Wall Width/2)
 (Actual Soil Bearing Pressure* Cantilever)
 (S = 12*h^2 / 4)
 (ACI 318-14 Equation 14.5.2.1 (a) & (b), ΦMn = min c
 and 0.9*0.85*(fc)*S / 1000*12)

Adequate in Flexure? **YES**

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 = **0.70 klf**
 ΦVn = **12.20 klf**

(Wu*Assumed Footing Span / 2)
 (ACI 318-14 Equation 22.5.5.1, ΦVn = 0.75*2*sqrt(fc

Adequate in One-Way Shear? **YES**

Are Stirrups Req'd? **NO**

Use #3 Stirrups @ 18 in. O.C.

(ACI 318-14 Section 7.6.3.1 If ΦVn/2 > Vu1 "No", Other
 (Provide minimum stirrups to support steel)

Wall Bearing Check:

$\Phi P_n = 69.62$ klf (ACI 318-14 Table 22.8.3.2 $\Phi P_n = 0.65 \cdot 0.85 \cdot f_c \cdot \text{Plate}$)
 Adequate in Bearing? **YES**

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

$M_u = 1.04$ k-ft	($W_u \cdot \text{Assumed Footing Span}^2 / 8$)
$m = 23.529$	($m = f_y / (0.85 \cdot f_c)$)
$R_u = 0.01$ ksi	($R_u = M_u / (0.9 \cdot B \cdot d^2)$)
$\rho \text{ Req'd} = 0.0002$	($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot R_u \cdot m / f_y})$)
$\rho \text{ Min.} = 0.0027$	(ACI 318-14 Equations 8.7.5.6.3.1 (a) and (b), Smaller)
$4/3 \cdot M_u \rho \text{ Req'd} = 0.0003$	($\rho = (1/m) \cdot (1 - \sqrt{1 - 2 \cdot 1.33 \cdot R_u \cdot m / f_y})$)
Governing $\rho = 0.0003$	(If $\rho \text{ Req'd} < 4/3 \cdot M_u \rho \text{ Req'd} < \rho \text{ Min.}$, Use $4/3 \cdot M_u \rho$)
A's Required = 0.04 sq.in.	($A_s = \text{Governing } \rho \cdot B \cdot d$)
Bar # = 4	
Number of Bars = 2 bars	
As Provided = 0.40 sq.in.	
Adequate Bott. Reinf. Provided? YES	

Top Reinforcement:

A's Required = 0.00 sq.in.
 Bar # = 4
 Number of Bars = 0 bars
 As Provided = 0.00 sq.in.
 Adequate Bott. Reinf. Provided? **YES**

Temperature & Shrinkage Reinforcement:

Minimum Steel = 0.39 sq.in. (ACI 318-14 Table 24.4.3.2 T&S Reinf. = $0.0018 \cdot 12$ in)
 As Provided Top = 0.00 sq.in.
 As Provided Bott = 0.40 sq.in.
 As Provided Total = 0.40 sq.in.
 T&S Steel Provided? **YES**

Design Checks:

Is $Q_{max} < SBC$? **YES**
 Adequate in One-Way Shear? **YES**
 Adequate in One-Way Shear (Span)? **YES**
 Adequate in Flexure? **YES**
 Adequate in Flexure (Span)? **YES**
 Adequate in Bearing? **YES**
 Adequate Top Reinf. Provided? **YES**
 Adequate Bott. Reinf. Provided? **YES**
 T&S Reinf. Provided? **YES**

Footing Design:	
Footing Width, B =	18.00 in.
Footing Depth, H =	12.00 in.
Top Steel =	() #4 bars
Bottom Steel =	(2) #4 bars
Stirrups =	#3 Stirrups @ 18 in. O.C.

Wind Load Distribution - Single Diaphragm Design, Envelope Method

General Building Info :

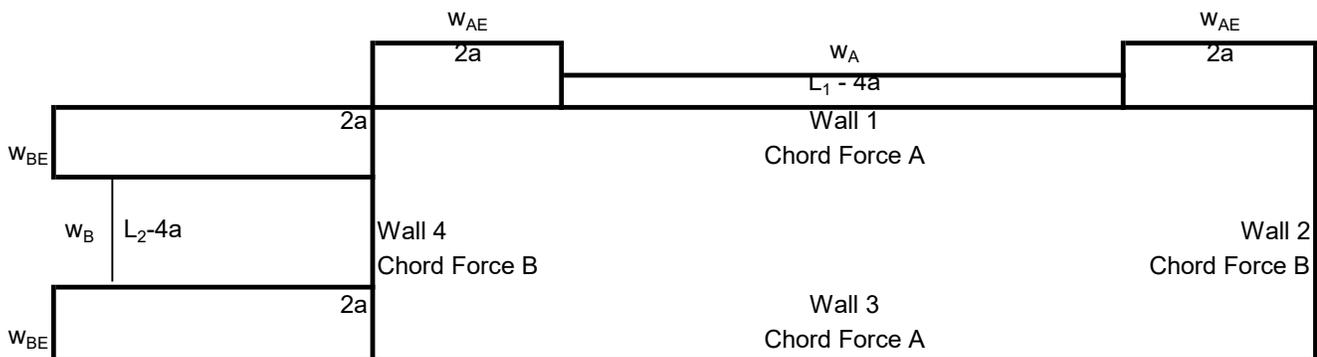
	Length	Wall Height to Roof	Parapet Height	Total Wall Height
Wall 1 =	76.67 ft	11.00 ft	0.00 ft	11.00 ft
Wall 2 =	28.67 ft	11.25 ft	1.75 ft	13.00 ft
Wall 3 =	76.67 ft	11.50 ft	1.50 ft	13.00 ft
Wall 4 =	28.67 ft	11.25 ft	1.75 ft	13.00 ft

General Wind Loading Info :

ASD Factor =	0.6	(1 for ACSE 7-05, 0.6 for ASCE 7-10)
2a =	6.00 ft	(2 x Length of End Zone per ACSE)
Zone 1 Pressure =	12.40 psf	(Refer to Tedd's Calculations for Pressures)
Zone 1E Pressure =	16.90 psf	
Zone 4 Pressure =	2.30 psf	
Zone 4E Pressure =	5.30 psf	
WW Parapet Pressure =	32.01 psf	
LW Parapet Pressure =	21.34 psf	

Wind Loading @ Roof:

$L_1 - 4a =$	64.67 ft	
Distributed Force, $w_A =$	131.97 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, $w_{AE} =$	174.34 plf	
$L_2 - 4a =$	16.67 ft	
Distributed Force, $w_B =$	176.05 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, $w_{BE} =$	218.24 plf	



Shear Wall Loads:

Wall 1 =	1.7 k ASD	x Factor =	2.8 k LRFD	(Distributed Load x Effective Trib)
Wall 2 =	3.2 k ASD	x Factor =	5.3 k LRFD	(Distributed Load x Effective Trib)
Wall 3 =	1.7 k ASD	x Factor =	2.8 k LRFD	(Distributed Load x Effective Trib)
Wall 4 =	3.2 k ASD	x Factor =	5.3 k LRFD	(Distributed Load x Effective Trib)

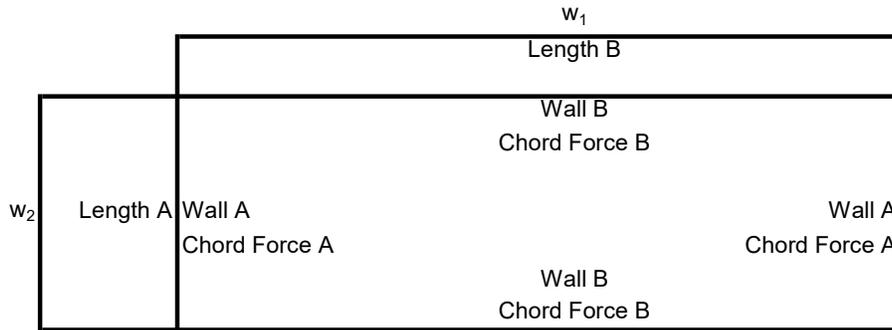
Diaphragm Loads:

Wall 1 =	21.7 plf ASD	x Factor =	36.2 plf LRFD	(Wall 1 Load*1000/Wall 1 Length)
Wall 2 =	111.2 plf ASD	x Factor =	185.3 plf LRFD	(Wall 2 Load*1000/Wall 2 Length)
Wall 3 =	21.7 plf ASD	x Factor =	36.2 plf LRFD	(Wall 3 Load*1000/Wall 3 Length)
Wall 4 =	111.2 plf ASD	x Factor =	185.3 plf LRFD	(Wall 4 Load*1000/Wall 4 Length)

Seismic Load Distribution - Single Diaphragm Design

General Info :

Seismic Response Coeff. C_s =	0.0162	(from Tedds Seismic Load Calculations)
Roof Dead Load =	15.00 psf	
Wall Length A =	28.67 ft	
Wall Weight A =	12.00 psf	
Wall Height A =	11.25 ft	(Roof to FFE)
Parapet Height A =	1.75 ft	(Top of Parapet to Roof)
Distributed Force, w_1 =	9.79 plf	$((\text{Length A} * \text{Dead} + (0.5 * \text{Height B} + \text{Parapet}) * \text{Weight B}) * C_s)$
Wall Length B =	76.67 ft	
Wall Weight B =	12.00 psf	
Wall Height B =	11.50 ft	(Roof to FFE)
Parapet Height B =	1.50 ft	(Top of Parapet to Roof)
Distributed Force, w_2 =	21.50 plf	$((\text{Length B} * \text{Dead} + (0.5 * \text{Height A} + \text{Parapet}) * \text{Weight A}) * C_s)$



Shear Wall Loads from Diaphragm:

Wall A =	0.3 k ASD	/0.7 =	0.4 k LRFD	$((0.7 * w_1 * \text{Length B} * 0.5) / 1000)$
Wall B =	0.2 k ASD	/0.7 =	0.3 k LRFD	$((0.7 * w_2 * \text{Length A} * 0.5) / 1000)$

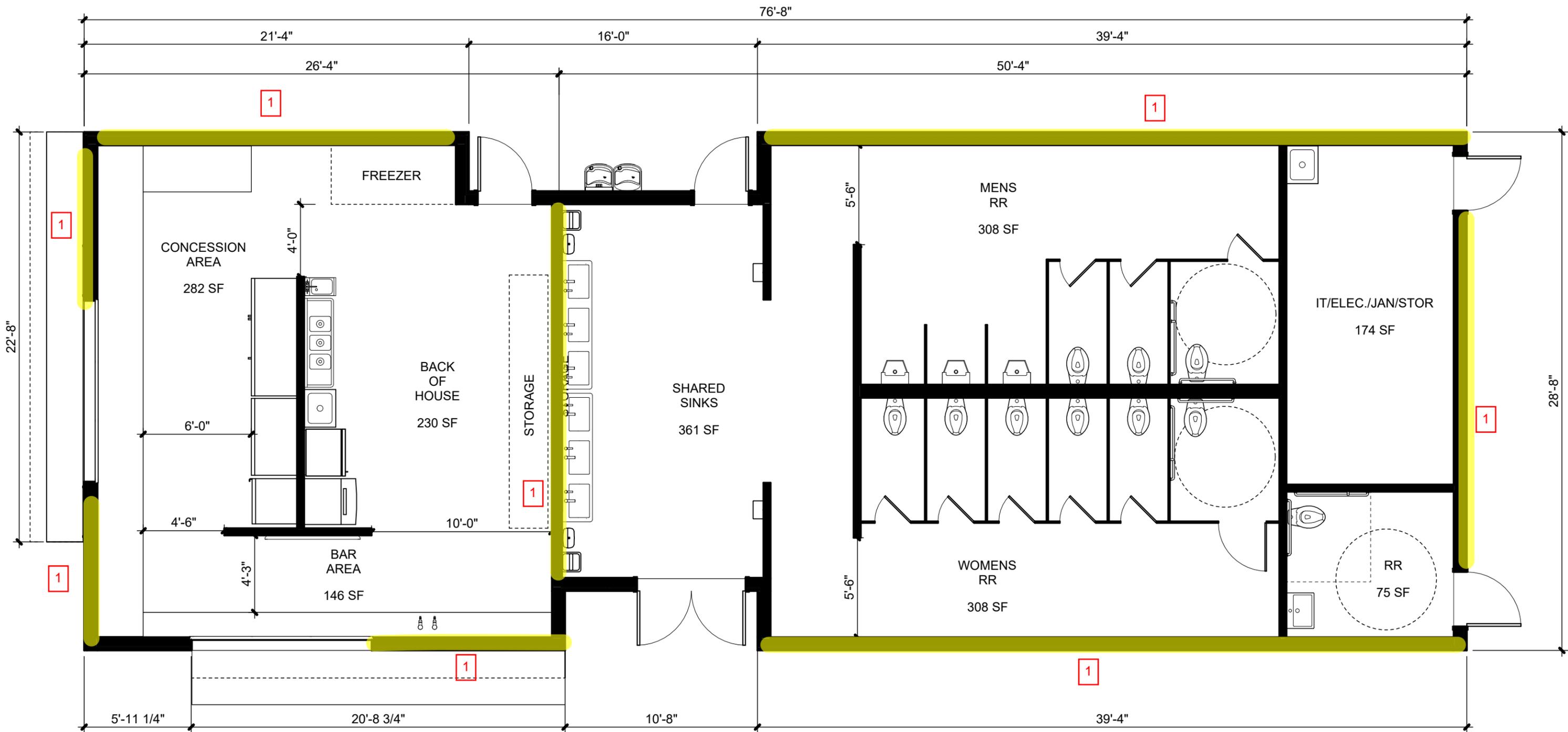
Total Shear Wall Loads (Includes Wall Weight):

Wall A =	0.3 k ASD	/0.7 =	0.4 k LRFD	$(\text{Dia. A} + \text{Length A} * \text{Weight A} * \text{Height A} * C_s)$
Wall B =	0.4 k ASD	/0.7 =	0.5 k LRFD	$(\text{Dia. B} + \text{Length B} * \text{Weight B} * \text{Height B} * C_s)$

Diaphragm Loads:

Wall A =	9.2 plf ASD	/0.7 =	13.1 plf LRFD	$(\text{Wall A Load} * 1000 / \text{Wall A Length})$
Wall B =	2.8 plf ASD	/0.7 =	4.0 plf LRFD	$(\text{Wall B Load} * 1000 / \text{Wall B Length})$

WIND CONTROLS



1 | CONCESSIONS + RESTROOMS FLOOR PLAN
 3/16" = 1'-0"

PARAGON STAR - RR CONCESSIONS

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.15

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7603	1680	0.221	PASS
Chord capacity	lb/in ²	1064	57	0.091	PASS
Deflection	in	0.270	0.095	0.353	PASS

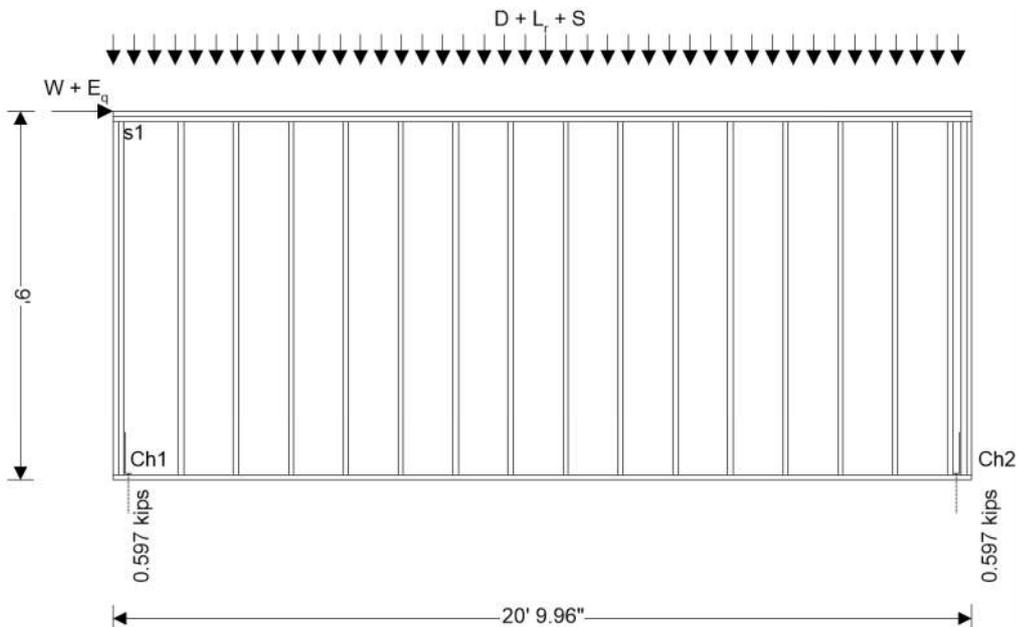
Panel details

Structural wood panel sheathing on one side

Panel height $h = 9$ ft

Panel length $b = 20.83$ ft

Total area of wall $A = h \times b = 187.47$ ft²



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	$A_s = 8.25$ in ²
Stud spacing	$s = 16$ in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	$A_e = 16.5$ in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	$A_{en} = 13.5$ in ²
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	$k_a = 34943$ lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	$G = 0.50$
Tension parallel to grain	$F_t = 575 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 1350 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 625 \text{ lb/in}^2$
Modulus of elasticity	$E = 1600000 \text{ lb/in}^2$
Minimum modulus of elasticity	$E_{min} = 580000 \text{ lb/in}^2$

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	$v_s = 520 \text{ lb/ft}$
Nominal unit shear capacity for wind design	$v_w = 730 \text{ lb/ft}$
Apparent shear wall shear stiffness	$G_a = 15 \text{ kips/in}$

Loading details

Dead load acting on top of panel	$D = 215 \text{ lb/ft}$
Roof live load acting on top of panel	$L_r = 287 \text{ lb/ft}$
Snow load acting on top of panel	$S = 201 \text{ lb/ft}$
Self weight of panel	$S_{wt} = 12 \text{ lb/ft}^2$
In plane wind load acting at head of panel	$W = 2800 \text{ lbs}$
Wind load serviceability factor	$f_{w\text{serv}} = 1.00$
In plane seismic load acting at head of panel	$E_q = 300 \text{ lbs}$
Design spectral response accel. par., short periods	$S_{DS} = 0.105$

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1	$D + 0.6W$
Load combination no.2	$D + 0.7E$
Load combination no.3	$D + 0.45W + 0.75L_f + 0.75(L_r \text{ or } S \text{ or } R)$
Load combination no.4	$D + 0.525E + 0.75L_f + 0.75S$
Load combination no.5	$0.6D + 0.6W$
Load combination no.6	$0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2	$C_D = 1.60$
Size factor for tension – Table 4A	$C_{Ft} = 1.30$
Size factor for compression – Table 4A	$C_{Fc} = 1.10$
Wet service factor for tension – Table 4A	$C_{Mt} = 1.00$
Wet service factor for compression – Table 4A	$C_{Mc} = 1.00$
Wet service factor for modulus of elasticity – Table 4A	$C_{ME} = 1.00$
Temperature factor for tension – Table 2.3.3	$C_{tt} = 1.00$
Temperature factor for compression – Table 2.3.3	$C_{tc} = 1.00$
Temperature factor for modulus of elasticity – Table 2.3.3	$C_{tE} = 1.00$
Incising factor – cl.4.3.8	$C_i = 1.00$
Buckling stiffness factor – cl.4.4.2	$C_T = 1.00$
Bearing area factor - cl. 3.10.4	$C_b = 1.0$
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T = 580000 \text{ psi}$

Critical buckling design value $F_{cE} = 0.822 \times E_{min} / (h / d)^2 = \mathbf{1236}$ psi
 Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = \mathbf{2376}$ psi
 For sawn lumber $c = \mathbf{0.8}$
 Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = \mathbf{0.45}$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
 Shear wall length $b = \mathbf{20.83}$ ft
 Shear wall aspect ratio $h / b = \mathbf{0.432}$

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w_max} = 0.6 \times W = \mathbf{1.68}$ kips
 Shear capacity for wind loading $V_w = v_w \times b / 2 = \mathbf{7.603}$ kips
 $V_{w_max} / V_w = \mathbf{0.221}$

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading $V_{s_max} = 0.7 \times E_q = \mathbf{0.21}$ kips
 Shear capacity for seismic loading $V_s = v_s \times b / 2 = \mathbf{5.416}$ kips
 $V_{s_max} / V_s = \mathbf{0.039}$

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio $h / b = \mathbf{0.432}$
 Load combination 5
 Shear force for maximum tension $V = 0.6 \times W = \mathbf{1.68}$ kips
 Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 = \mathbf{0.129}$ kips
 Maximum tensile force in chord $T = V \times h / b - P = \mathbf{0.597}$ kips
 Maximum applied tensile stress $f_t = T / A_{en} = \mathbf{44}$ lb/in²
 Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in²
 $f_t / F_t' = \mathbf{0.037}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1
 Shear force for maximum compression $V = 0.6 \times W = \mathbf{1.68}$ kips
 Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.215}$ kips
 Maximum compressive force in chord $C = V \times h / b + P = \mathbf{0.941}$ kips
 Maximum applied compressive stress $f_c = C / A_e = \mathbf{57}$ lb/in²
 Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1064}$ lb/in²
 $f_c / F_c' = \mathbf{0.054}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c_perp}' = F_{c_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in²
 $f_c / F_{c_perp}' = \mathbf{0.091}$

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 $T_1 = \mathbf{0.597}$ kips
 Chord 2 $T_2 = \mathbf{0.597}$ kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q_ch[i]R}$ (lbs)	$D_{C_ch[i]R}$ (lbs)	$D_{T_ch[i]R}$ (lbs)	$L_{f_ch[i]R}$ (lbs)	$L_{r_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)

Ch1	-1210;	-130;	215;	215;	0;	191;	134;	0;
Ch2	1210;	130;	215;	215;	0;	191;	134;	0;

Wind load deflection

Design shear force

$$V_{\delta w} = f_{w_{serv}} \times W = \mathbf{2.8 \text{ kips}}$$

Deflection limit

$$\Delta_{w_allow} = h / 400 = \mathbf{0.27 \text{ in}}$$

Induced unit shear

$$v_{\delta w} = V_{\delta w} / b = \mathbf{134.42 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) = \mathbf{1.081 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.031 \text{ in}}$$

Shear wall deflection – Eqn. 4.3-1

$$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.095 \text{ in}}$$

$$\delta_{sww} / \Delta_{w_allow} = \mathbf{0.353}$$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force

$$V_{\delta s} = E_q = \mathbf{0.3 \text{ kips}}$$

Deflection limit

$$\Delta_{s_allow} = 0.020 \times h = \mathbf{2.16 \text{ in}}$$

Induced unit shear

$$v_{\delta s} = V_{\delta s} / b = \mathbf{14.4 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{Ds}) \times (D + S_{wt} \times h) \times s / 2) = \mathbf{0.005 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.000 \text{ in}}$$

Shear wall elastic deflection – Eqn. 4.3-1

$$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.009 \text{ in}}$$

Deflection amplification factor

$$C_{d\delta} = \mathbf{4}$$

Seismic importance factor

$$I_e = \mathbf{1.25}$$

Amp. seis. deflection – ASCE 7-10, Eqn.12.8-15

$$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.028 \text{ in}}$$

$$\delta_{sws} / \Delta_{s_allow} = \mathbf{0.013}$$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.15

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	6935	3180	0.459	PASS
Chord capacity	lb/in ²	1064	97	0.155	PASS
Deflection	in	0.270	0.204	0.755	PASS

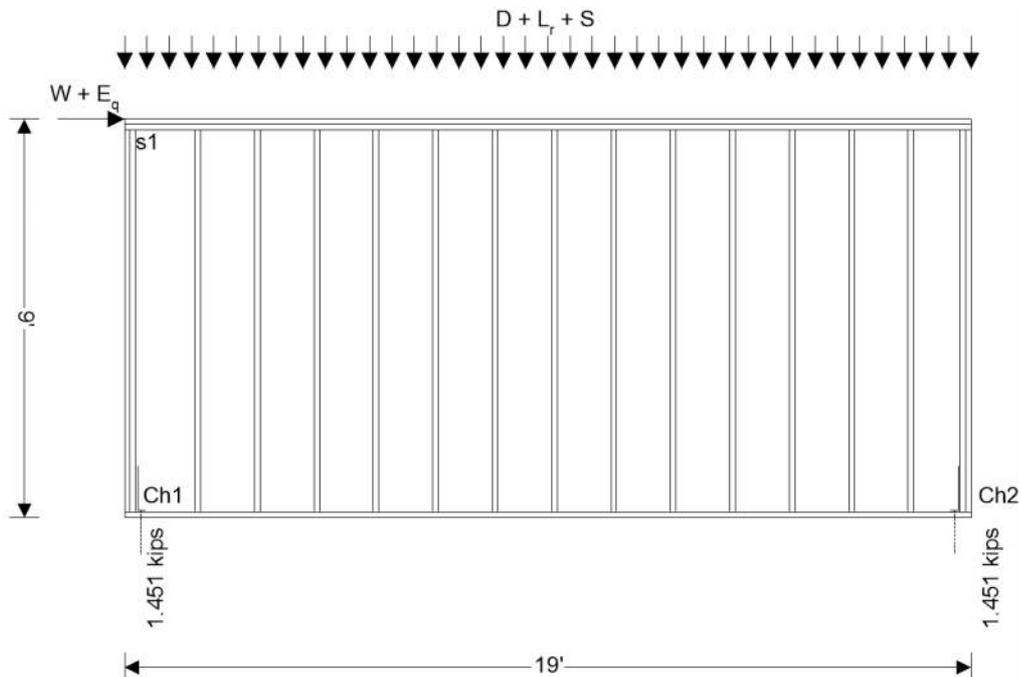
Panel details

Structural wood panel sheathing on one side

Panel height $h = 9$ ft

Panel length $b = 19$ ft

Total area of wall $A = h \times b = 171$ ft²



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	$A_s = 8.25$ in ²
Stud spacing	$s = 16$ in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	$A_e = 16.5$ in ²
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	$A_{en} = 13.5$ in ²
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	$k_a = 34943$ lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	$G = 0.50$
Tension parallel to grain	$F_t = 575 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 1350 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 625 \text{ lb/in}^2$
Modulus of elasticity	$E = 1600000 \text{ lb/in}^2$
Minimum modulus of elasticity	$E_{min} = 580000 \text{ lb/in}^2$

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	$v_s = 520 \text{ lb/ft}$
Nominal unit shear capacity for wind design	$v_w = 730 \text{ lb/ft}$
Apparent shear wall shear stiffness	$G_a = 15 \text{ kips/in}$

Loading details

Dead load acting on top of panel	$D = 30 \text{ lb/ft}$
Roof live load acting on top of panel	$L_r = 40 \text{ lb/ft}$
Snow load acting on top of panel	$S = 60 \text{ lb/ft}$
Self weight of panel	$S_{wt} = 12 \text{ lb/ft}^2$
In plane wind load acting at head of panel	$W = 5300 \text{ lbs}$
Wind load serviceability factor	$f_{w\text{serv}} = 1.00$
In plane seismic load acting at head of panel	$E_q = 400 \text{ lbs}$
Design spectral response accel. par., short periods	$S_{DS} = 0.105$

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1	$D + 0.6W$
Load combination no.2	$D + 0.7E$
Load combination no.3	$D + 0.45W + 0.75L_f + 0.75(L_r \text{ or } S \text{ or } R)$
Load combination no.4	$D + 0.525E + 0.75L_f + 0.75S$
Load combination no.5	$0.6D + 0.6W$
Load combination no.6	$0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2	$C_D = 1.60$
Size factor for tension – Table 4A	$C_{Ft} = 1.30$
Size factor for compression – Table 4A	$C_{Fc} = 1.10$
Wet service factor for tension – Table 4A	$C_{Mt} = 1.00$
Wet service factor for compression – Table 4A	$C_{Mc} = 1.00$
Wet service factor for modulus of elasticity – Table 4A	$C_{ME} = 1.00$
Temperature factor for tension – Table 2.3.3	$C_{tt} = 1.00$
Temperature factor for compression – Table 2.3.3	$C_{tc} = 1.00$
Temperature factor for modulus of elasticity – Table 2.3.3	$C_{tE} = 1.00$
Incising factor – cl.4.3.8	$C_i = 1.00$
Buckling stiffness factor – cl.4.4.2	$C_T = 1.00$
Bearing area factor - cl. 3.10.4	$C_b = 1.0$
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T = 580000 \text{ psi}$

Critical buckling design value $F_{cE} = 0.822 \times E_{min} / (h / d)^2 = \mathbf{1236}$ psi
 Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = \mathbf{2376}$ psi
 For sawn lumber $c = \mathbf{0.8}$
 Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = \mathbf{0.45}$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
 Shear wall length $b = \mathbf{19}$ ft
 Shear wall aspect ratio $h / b = \mathbf{0.474}$

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w_max} = 0.6 \times W = \mathbf{3.18}$ kips
 Shear capacity for wind loading $V_w = V_w \times b / 2 = \mathbf{6.935}$ kips
 $V_{w_max} / V_w = \mathbf{0.459}$

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading $V_{s_max} = 0.7 \times E_q = \mathbf{0.28}$ kips
 Shear capacity for seismic loading $V_s = V_s \times b / 2 = \mathbf{4.94}$ kips
 $V_{s_max} / V_s = \mathbf{0.057}$

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio $h / b = \mathbf{0.474}$
 Load combination 5
 Shear force for maximum tension $V = 0.6 \times W = \mathbf{3.18}$ kips
 Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 = \mathbf{0.055}$ kips
 Maximum tensile force in chord $T = V \times h / b - P = \mathbf{1.451}$ kips
 Maximum applied tensile stress $f_t = T / A_{en} = \mathbf{107}$ lb/in²
 Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in²
 $f_t / F_t' = \mathbf{0.090}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1
 Shear force for maximum compression $V = 0.6 \times W = \mathbf{3.18}$ kips
 Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.092}$ kips
 Maximum compressive force in chord $C = V \times h / b + P = \mathbf{1.598}$ kips
 Maximum applied compressive stress $f_c = C / A_e = \mathbf{97}$ lb/in²
 Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1064}$ lb/in²
 $f_c / F_c' = \mathbf{0.091}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c_perp}' = F_{c_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in²
 $f_c / F_{c_perp}' = \mathbf{0.155}$

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 $T_1 = \mathbf{1.451}$ kips
 Chord 2 $T_2 = \mathbf{1.451}$ kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q_ch[i]R}$ (lbs)	$D_{C_ch[i]R}$ (lbs)	$D_{T_ch[i]R}$ (lbs)	$L_{f_ch[i]R}$ (lbs)	$L_{r_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)

Ch1	-2511;	-189;	92;	92;	0;	27;	40;	0;
Ch2	2511;	189;	92;	92;	0;	27;	40;	0;

Wind load deflection

Design shear force

$$V_{\delta w} = f_{w_{serv}} \times W = \mathbf{5.3 \text{ kips}}$$

Deflection limit

$$\Delta_{w_allow} = h / 400 = \mathbf{0.27 \text{ in}}$$

Induced unit shear

$$v_{\delta w} = V_{\delta w} / b = \mathbf{278.95 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) = \mathbf{2.455 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.070 \text{ in}}$$

Shear wall deflection – Eqn. 4.3-1

$$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.204 \text{ in}}$$

$$\delta_{sww} / \Delta_{w_allow} = \mathbf{0.755}$$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force

$$V_{\delta s} = E_q = \mathbf{0.4 \text{ kips}}$$

Deflection limit

$$\Delta_{s_allow} = 0.020 \times h = \mathbf{2.16 \text{ in}}$$

Induced unit shear

$$v_{\delta s} = V_{\delta s} / b = \mathbf{21.05 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{Ds}) \times (D + S_{wt} \times h) \times s / 2) = \mathbf{0.136 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.004 \text{ in}}$$

Shear wall elastic deflection – Eqn. 4.3-1

$$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.015 \text{ in}}$$

Deflection amplification factor

$$C_{d\delta} = \mathbf{4}$$

Seismic importance factor

$$I_e = \mathbf{1.25}$$

Amp. seis. deflection – ASCE 7-10, Eqn.12.8-15

$$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.047 \text{ in}}$$

$$\delta_{sws} / \Delta_{s_allow} = \mathbf{0.022}$$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.15

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	3314	1590	0.480	PASS
Chord capacity	lb/in ²	1064	101	0.162	PASS
Deflection	in	0.270	0.255	0.945	PASS

Panel details

Structural wood panel sheathing on one side

Panel height

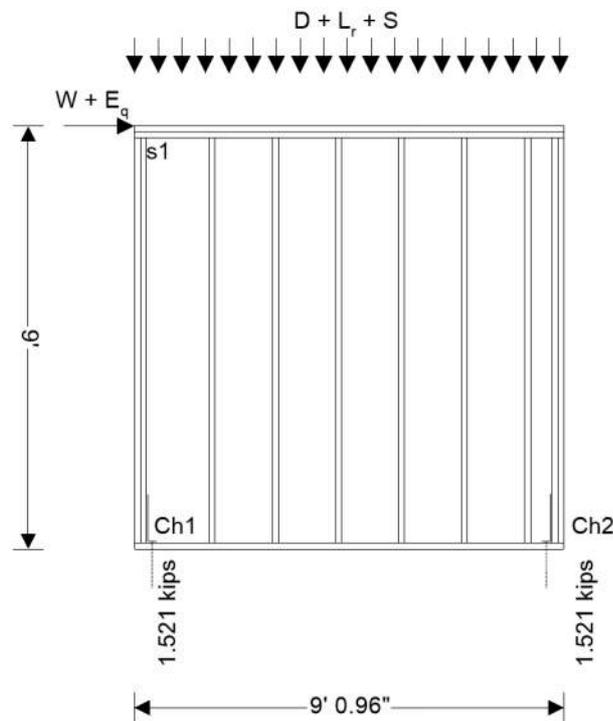
$$h = 9 \text{ ft}$$

Panel length

$$b = 9.08 \text{ ft}$$

Total area of wall

$$A = h \times b = 81.72 \text{ ft}^2$$



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	$A_s = 8.25 \text{ in}^2$
Stud spacing	$s = 16 \text{ in}$
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	$A_e = 16.5 \text{ in}^2$
Hole diameter	Dia = 1 in
Net cross-sectional area of end posts	$A_{en} = 13.5 \text{ in}^2$
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry

Temperature 100 degF or less
 Vertical anchor stiffness $K_a = 34943$ lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.2 grade, 2" & wider
 Specific gravity $G = 0.50$
 Tension parallel to grain $F_t = 575$ lb/in²
 Compression parallel to grain $F_c = 1350$ lb/in²
 Compression perpendicular to grain $F_{c_perp} = 625$ lb/in²
 Modulus of elasticity $E = 1600000$ lb/in²
 Minimum modulus of elasticity $E_{min} = 580000$ lb/in²

Sheathing details

Sheathing material 7/16" wood panel oriented strandboard sheathing
 Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design $V_s = 520$ lb/ft
 Nominal unit shear capacity for wind design $V_w = 730$ lb/ft
 Apparent shear wall shear stiffness $G_a = 15$ kips/in

Loading details

Dead load acting on top of panel $D = 30$ lb/ft
 Roof live load acting on top of panel $L_r = 40$ lb/ft
 Snow load acting on top of panel $S = 60$ lb/ft
 Self weight of panel $S_{wt} = 12$ lb/ft²
 In plane wind load acting at head of panel $W = 2650$ lbs
 Wind load serviceability factor $f_{w serv} = 1.00$
 In plane seismic load acting at head of panel $E_q = 200$ lbs
 Design spectral response accel. par., short periods $S_{DS} = 0.105$

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1 $D + 0.6W$
 Load combination no.2 $D + 0.7E$
 Load combination no.3 $D + 0.45W + 0.75L_f + 0.75(L_r \text{ or } S \text{ or } R)$
 Load combination no.4 $D + 0.525E + 0.75L_f + 0.75S$
 Load combination no.5 $0.6D + 0.6W$
 Load combination no.6 $0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2 $C_D = 1.60$
 Size factor for tension – Table 4A $C_{Ft} = 1.30$
 Size factor for compression – Table 4A $C_{Fc} = 1.10$
 Wet service factor for tension – Table 4A $C_{Mt} = 1.00$
 Wet service factor for compression – Table 4A $C_{Mc} = 1.00$
 Wet service factor for modulus of elasticity – Table 4A
 $C_{ME} = 1.00$
 Temperature factor for tension – Table 2.3.3 $C_{tt} = 1.00$
 Temperature factor for compression – Table 2.3.3
 $C_{tc} = 1.00$
 Temperature factor for modulus of elasticity – Table 2.3.3
 $C_{tE} = 1.00$
 Incising factor – cl.4.3.8 $C_i = 1.00$
 Buckling stiffness factor – cl.4.4.2 $C_T = 1.00$

Bearing area factor - cl. 3.10.4 $C_b = 1.0$
 Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{TE} \times C_i \times C_T = 580000$ psi
 Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 1236$ psi
 Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = 2376$ psi
 For sawn lumber $c = 0.8$
 Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.45$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
 Shear wall length $b = 9.08$ ft
 Shear wall aspect ratio $h / b = 0.991$

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w_max} = 0.6 \times W = 1.59$ kips
 Shear capacity for wind loading $V_w = v_w \times b / 2 = 3.314$ kips
 $V_{w_max} / V_w = 0.48$
PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading $V_{s_max} = 0.7 \times E_q = 0.14$ kips
 Shear capacity for seismic loading $V_s = v_s \times b / 2 = 2.361$ kips
 $V_{s_max} / V_s = 0.059$
PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio $h / b = 0.991$
 Load combination 5
 Shear force for maximum tension $V = 0.6 \times W = 1.59$ kips
 Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 = 0.055$ kips
 Maximum tensile force in chord $T = V \times h / b - P = 1.521$ kips
 Maximum applied tensile stress $f_t = T / A_{en} = 113$ lb/in²
 Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1196$ lb/in²
 $f_t / F_t' = 0.094$
PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1
 Shear force for maximum compression $V = 0.6 \times W = 1.59$ kips
 Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 = 0.092$ kips
 Maximum compressive force in chord $C = V \times h / b + P = 1.668$ kips
 Maximum applied compressive stress $f_c = C / A_e = 101$ lb/in²
 Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 1064$ lb/in²
 $f_c / F_c' = 0.095$
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c_perp}' = F_{c_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = 625$ lb/in²
 $f_c / F_{c_perp}' = 0.162$
PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 $T_1 = 1.521$ kips
 Chord 2 $T_2 = 1.521$ kips

Chord reactions by load type

Chord	$W_{ch[i]R}$	$E_{q_ch[i]R}$	$DC_{ch[i]R}$	$DT_{ch[i]R}$	$Lf_{ch[i]R}$	$Lr_{ch[i]R}$	$Sch[i]R$	$Rch[i]R$
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	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
Ch1	-2627;	-198;	92;	92;	0;	27;	40;	0;
Ch2	2627;	198;	92;	92;	0;	27;	40;	0;

Wind load deflection

Design shear force

$$V_{\delta w} = f_{Wserv} \times W = \mathbf{2.65 \text{ kips}}$$

Deflection limit

$$\Delta_{w_allow} = h / 400 = \mathbf{0.27 \text{ in}}$$

Induced unit shear

$$v_{\delta w} = V_{\delta w} / b = \mathbf{291.85 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) = \mathbf{2.571 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.074 \text{ in}}$$

Shear wall deflection – Eqn. 4.3-1

$$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.255 \text{ in}}$$

$$\delta_{sww} / \Delta_{w_allow} = \mathbf{0.945}$$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force

$$V_{\delta s} = E_q = \mathbf{0.2 \text{ kips}}$$

Deflection limit

$$\Delta_{s_allow} = 0.020 \times h = \mathbf{2.16 \text{ in}}$$

Induced unit shear

$$v_{\delta s} = V_{\delta s} / b = \mathbf{22.03 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{Ds}) \times (D + S_{wt} \times h) \times s / 2) = \mathbf{0.145 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.004 \text{ in}}$$

Shear wall elastic deflection – Eqn. 4.3-1

$$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.018 \text{ in}}$$

Deflection amplification factor

$$C_{d\delta} = \mathbf{4}$$

Seismic importance factor

$$I_e = \mathbf{1.25}$$

Amp. seis. deflection – ASCE 7-10, Eqn.12.8-15

$$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.057 \text{ in}}$$

$$\delta_{sws} / \Delta_{s_allow} = \mathbf{0.026}$$

PASS - Shear wall deflection is less than deflection limit



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Company:		Page:	1
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Shear Wall Holddown	Date:	11/14/2025
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 5/8	
Item number:	2198027 HAS-V-36 5/8"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)	
Specification text:	Hilti Ø 5/8 in HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) with 10 in nominal embedment depth per ICC-ES ESR-4868 , Hammer drill bit installation per MPII	
Effective embedment depth:	$h_{ef,act} = 10.000$ in. ($h_{ef,limit} = -$ in.)	
Material:	ASTM F1554 Grade 36	
Evaluation Service Report:	ESR-4868	
Issued Valid:	11/1/2024 11/1/2026	
Proof:	Design Method ACI 318-19 / Chem	
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)	
Stand-off installation:		
Profile:		
Base material:	cracked concrete, 3000, $f'_c = 3,000$ psi; $h = 34.000$ in., Temp. short/long: 32/32 °F	
Installation:	Hammer drilled hole, Installation condition: Dry	
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar	

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Company:
Address:
Phone | Fax: |
Design: Shear Wall Holddown
Fastening point:

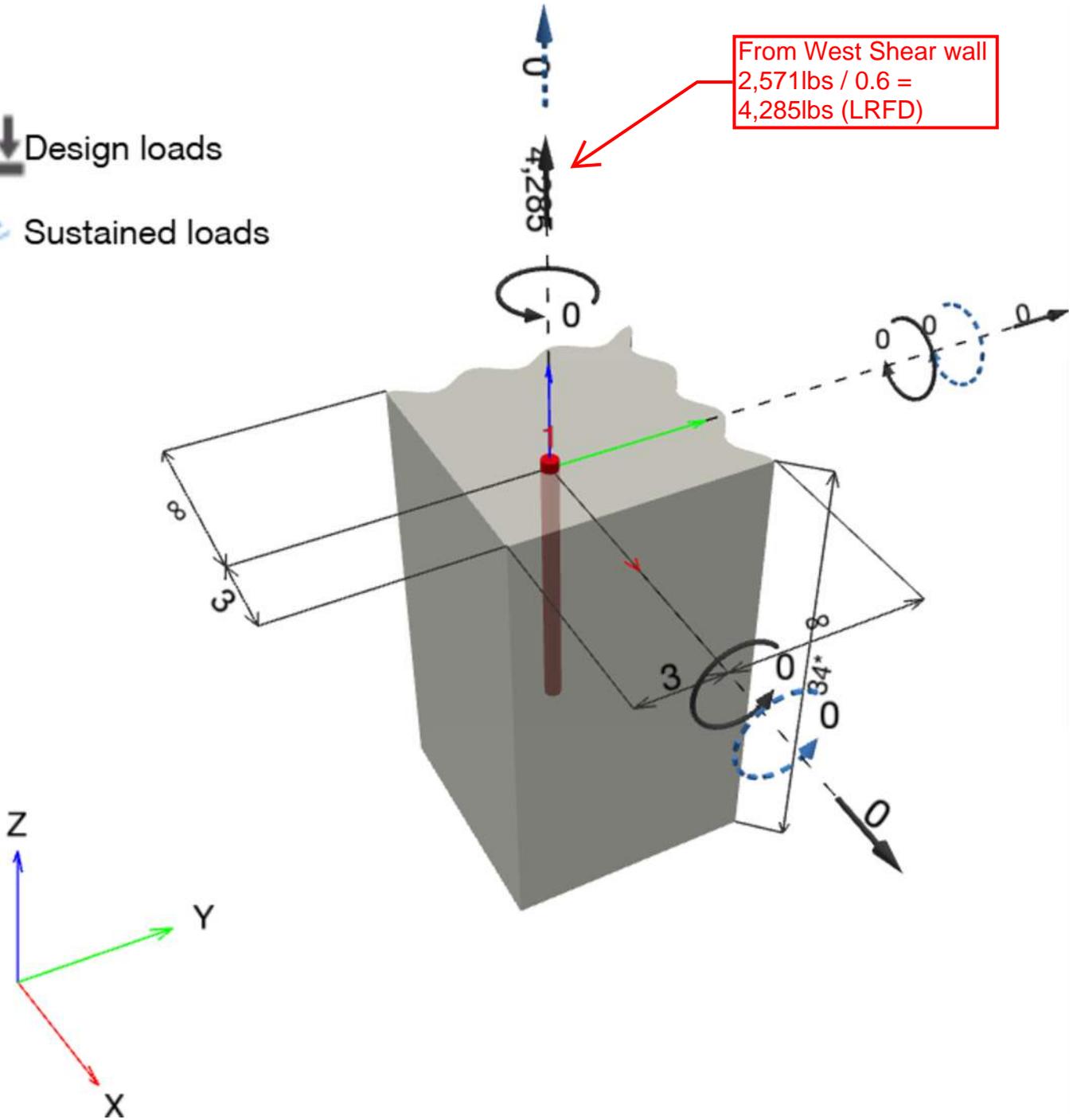
Page: 2
Specifier:
E-Mail:
Date: 11/14/2025

Geometry [in.] & Loading [lb, in.lb]

Design loads

Sustained loads

From West Shear wall
 $2,571\text{lbs} / 0.6 =$
 $4,285\text{lbs (LRFD)}$



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Shear Wall Holddown	Date:	11/14/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,285; V _x = 0; V _y = 0; M _x = 0; M _y = 0; M _z = 0;	no	82



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Company:		Page:	4
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Shear Wall Holddown	Date:	11/14/2025
Fastening point:			

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb]		Utilization	Status
		Load	Capacity	β_N / β_V [%]	
Tension	Concrete Breakout Failure	4,285	5,236	82 / -	OK
Shear	-	-	-	- / -	N/A

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	-	-	-	-	N/A

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!